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Papers

Journal of the
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# TOLKMITT'S BACKWATER AND DROPDOWN CURVE TABLES

By R. D. Goodrich,[1] F. ASCE

## SYNOPSIS

Tables and formulas are presented for the computation of backwater and opdown curves for channels of parabolic section. These are followed by utions as to the selection of the coefficients C and n when used in connecon with the computation of water-surface curves.

## INTRODUCTION

The integration of equations for varied flow in natural and artificial chanls continues to be of interest to hydraulic engineers. This is evidenced by e many papers published in the Transactions and Proceedings of the ASCE, gether with their applications in the determination of backwater and dropwn curves; an excellent list of references and investigations was given by n Te Chow in 1955.[2] Several papers have been published on these subjects nce 1955, among which is that by Joe M. Lara and Kenneth B. Schroder.[3]

Most of the tables available for the computation of water-surface curves ve been developed for channels of rectangular, trapezoidal, and circular oss section.[2] The formulas and tables by G. Tolkmitt[4] were developed for

Note.—Discussion open until October 1, 1960. To extend the closing date one month, ritten request must be filed with the Executive Secretary, ASCE. This paper is part the copyrighted Journal of the Hydraulics Division, Proceedings of the American iety of Civil Engineers, Vol. 86, No. HY 5, May, 1960.

1 Chf. Engr., Cons. Engr., Grand Junction, Colo.

2 "Integrating the Equation of Gradually Varied Flow," by Ven Te Chow, Proceeds, ASCE, No. 838, November, 1955.

3 "Two Methods to Compute Water Surface Profiles," by Joe M. Lara and Kenneth Schroder, Proceedings, ASCE, Part I, Vol. 85, No. HY 4, April, 1959.

4 "Wasserbaukunst," (River Engineering), by G. Tolkmitt, Ernst and Sohn, Berlin, 8.

channels of parabolic cross section and to the writer's knowledge are only be found in the German texts by that author.

## FORMULAS AND TABLES

In 1932, Boris A. Bakhmeteff[5] discussed Tolkmitt's formulas only briefl Bakhmeteff's formulas and tables are intended to be applicable to channels any type, although in many cases they are not easily applied. The tables Bakhmeteff have been recomputed and extended by Chow,[2] and their use illu trated for rectangular, trapezoidal, and circular channels. Since it appea that Tolkmitt's tables may fill a gap in the material available as an aid to t computation of backwater curves, and although admittedly the gap is not a ser ous one, they are submitted herewith. The tables were furnished some yea ago by H. van der Veen, then a member of the Chihli River Commission. also supplied the following notes with the formulas for the backwater and dro down curves:

> Backwater curve caused by the raising of the water level at a certain point of a river. (For example, by a weir, dam or sluice gates.)
>
> Tolkmitt (Wasserbaukunst, River Engineering) gives for the calculation of the distance over which a certain raising of the water level at a certain point of a river affects the water level above that point, the following formula:

$$L_z = \frac{d}{s}\left[f\left(\frac{d+h}{d}\right) - f\left(\frac{d+z}{d}\right)\right] \quad \ldots\ldots\ldots\ldots \quad ($$

in which $L_z$ = distance at which raising of water level is z; h = raising of wat level causing the backwater curve; d = original depth of section at the stru ture; f = function; s = slope or fall per (unit) length; C = coefficient in formu $v = C\sqrt{R\,s}$; and

$$f\left(\frac{d+z}{d}\right) = \frac{d+z}{d} - \frac{1}{4}\log\left(1+2\frac{d}{z}\right) - \frac{1}{2}\tan^{-1}\left(1+\frac{z}{d}\right) + \frac{\pi}{4} \quad \ldots \quad ($$

> Backwater curve caused by the sudden lowering of the water level at a certain point of a river. (For example, cuttings, the removal of rapids, the opening of weirs, dykes, etc.)
>
> Tolkmitt (Wasserbaukunst, River Engineering) gives for the caclulation of the distance over which a certain lowering of the water level at a certain point of a river affects the water level; above that point, the following formula;

$$L_z = \frac{d}{s}\left[f\left(\frac{d-z}{d}\right)\right] - f\left(\frac{d-h}{d}\right)\left(1 - \frac{sC^2}{g}\right) - \left(\frac{h-z}{s}\right) \quad \ldots\ldots\ldots \quad ($$

in which Lz = distance at which the lowering of the water level is z; h = th

5 "Hydraulics of Open Channels," by Boris A. Bakhmeteff, McGraw-Hill Book Co Inc. Ed. 1932.

vering of the water level causing the backwater curve; d = original mean pth of the river; f = function; s = slope or fall per (unit) length; C = coeffi- ent in formula $v = C\sqrt{R\ s}$; and

$$f\left(\frac{d-z}{d}\right) = \frac{1}{4}\log\left(\frac{2d}{z} - 1\right) + \frac{1}{2}\tan^{-1}\left(\frac{d-z}{d}\right) \quad \ldots\ldots\ldots \quad (4)$$

The tables were entirely recomputed and enlarged by the addition of twenty ms in each table. The formulas were modified[a] in order to facilitate the mputation, and the new computations were carried out to six decimals, then unded to four places to correspond with the usual tables of Bresse's func- ns for rectangular channels. The results are given in Tables 1 and 2, and

TABLE 1

| | f(X) | X | f(X) | X | f(X) | X | f(X) |
|---|---|---|---|---|---|---|---|
| 00 | -∞ | 1.075 | 0.6196 | 1.31 | 1.1339 | 1.85 | 1.7953 |
| 01 | -0.5069 | 1.080 | 0.6390 | 1.32 | 1.1489 | 1.90 | 1.8497 |
| 02 | -0.3330 | 1.085 | 0.6574 | 1.33 | 1.1637 | 1.95 | 1.9037 |
| 03 | -0.2310 | 1.090 | 0.6749 | 1.34 | 1.1783 | 2.00 | 1.9572 |
| 04 | -0.1585 | 1.095 | 0.6917 | 1.35 | 1.1927 | 2.1 | 2.0632 |
| 05 | -0.1020 | 1.10 | 0.7078 | 1.36 | 1.2069 | 2.2 | 2.1681 |
| 06 | -0.0558 | 1.11 | 0.7382 | 1.37 | 1.2210 | 2.3 | 2.2722 |
| 07 | -0.0167 | 1.12 | 0.7665 | 1.38 | 1.2349 | 2.4 | 2.3756 |
| 08 | 0.0173 | 1.13 | 0.7931 | 1.39 | 1.2486 | 2.5 | 2.4784 |
| 09 | 0.0474 | 1.14 | 0.8183 | 1.40 | 1.2622 | 2.6 | 2.5809 |
| 10 | 0.0744 | 1.15 | 0.8422 | 1.41 | 1.2756 | 2.7 | 2.6829 |
| 12 | 0.1212 | 1.16 | 0.8651 | 1.42 | 1.2890 | 2.8 | 2.7847 |
| 14 | 0.1610 | 1.17 | 0.8869 | 1.43 | 1.3022 | 2.9 | 2.8862 |
| 16 | 0.1957 | 1.18 | 0.9080 | 1.44 | 1.3153 | 3.0 | 2.9876 |
| 18 | 0.2264 | 1.19 | 0.9283 | 1.45 | 1.3282 | 3.5 | 3.4922 |
| 20 | 0.2540 | 1.20 | 0.9479 | 1.46 | 1.3411 | 4.0 | 3.9948 |
| 25 | 0.3129 | 1.21 | 0.9670 | 1.47 | 1.3539 | 4.5 | 4.4963 |
| 30 | 0.3617 | 1.22 | 0.9854 | 1.48 | 1.3666 | 5.0 | 4.9973 |
| 35 | 0.4034 | 1.23 | 1.0034 | 1.49 | 1.3791 | 6.0 | 5.9985 |
| 40 | 0.4399 | 1.24 | 1.0209 | 1.50 | 1.3916 | 7.0 | 6.9990 |
| 45 | 0.4726 | 1.25 | 1.0382 | 1.55 | 1.4530 | 8.0 | 7.9994 |
| 50 | 0.5021 | 1.26 | 1.0548 | 1.60 | 1.5127 | 9.0 | 8.9996 |
| 55 | 0.5291 | 1.27 | 1.0712 | 1.65 | 1.5711 | 10 | 9.9997 |
| 60 | 0.5541 | 1.28 | 1.0873 | 1.70 | 1.6284 | 15 | 14.9999 |
| 65 | 0.5773 | 1.29 | 1.1031 | 1.75 | 1.6848 | 20 | 20.0000 |
| 70 | 0.5991 | 1.30 | 1.1186 | 1.80 | 1.7404 | ∞ | ∞ |

appropriate backwater and dropdown sketches are presented in Figs. 1 and In Table 1, Eqs. 1 and 2 were used with X = (d + z)/d and in Table 2, Eqs. 3 4 were used with X = (d - z)/d. In Table 2 if the functions are founded to ecimals, those values that have a horizontal bar over the 5 in the fourth imal place should have the number in the third decimal place increased.

The use of tables in the computation of backwater and dropdown curves has n fully illustrated and explained elsewhere and, therefore, there is no dis- sion of this procedure included herein. However, some suggestions and tions in the application of formulas in this type of work are offered.

[a] The formulas were rearranged for more convenient recomputation of the tables as wn below each table.

The Manning formula has come into very general use, replacing the Chez Kutter formula to a very large extent. The Chezy-Kutter formula, howeve was the one in general use at the time the Bresse and Tolkmitt tables we published. To use the coefficient C with the Manning formula, Chesley Posey[6] has proposed the formula:

$$C = \frac{1.49\ R^{2/3}}{n\ y^{1/2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \ ($$

with

$$v = C\ y^{1/2}\ s^{1/2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \ ($$

In applying Eq. 5, y is to be taken as the characteristic depth of the section; parabolic, it would be the depth at the vertex of the theoretical curve. T other symbols have the usual connotation.

TABLE 2

| X | f(X) | X | f(X) | X | f(X) | X | f(X |
|---|---|---|---|---|---|---|---|
| 1.000 | ∞ | .968 | 1.4143 | .80 | .8867 | .48 | .48 |
| .999 | 2.2925 | .966 | 1.3984 | .79 | .8700 | .46 | .46 |
| .998 | 2.1189 | .964 | 1.3833 | .78 | .8539 | .44 | .44 |
| .997 | 2.0171 | .962 | 1.3690 | .77 | .8383 | .42 | .42 |
| .996 | 1.9448 | .960 | 1.3555 | .76 | .8230 | .40 | .40 |
| .995 | 1.8887 | .955 | 1.3239 | .75 | .8082 | .38 | .38 |
| .994 | 1.8427 | .950 | 1.2958 | .74 | .7938 | .36 | .36 |
| .993 | 1.8038 | .945 | 1.2700 | .73 | .7797 | .34 | .34 |
| .992 | 1.7701 | .940 | 1.2463 | .72 | .7656 | .32 | .32 |
| .991 | 1.7402 | .935 | 1.2243 | .71 | .7523 | .30 | .30 |
| .990 | 1.7135 | .930 | 1.2035 | .70 | .7390 | .28 | .28 |
| .989 | 1.6893 | .925 | 1.1845 | .69 | .7260 | .26 | .26 |
| .988 | 1.6672 | .920 | 1.1664 | .68 | .7131 | .24 | .24 |
| .987 | 1.6468 | .915 | 1.1490 | .67 | .7005 | .22 | .22 |
| .986 | 1.6279 | .910 | 1.1329 | .66 | .6881 | .20 | .20 |
| .985 | 1.6103 | .905 | 1.1174 | .65 | .6758 | .18 | .18 |
| .984 | 1.5937 | .900 | 1.1023 | .64 | .6637 | .16 | .16 |
| .983 | 1.5782 | .89 | 1.0746 | .63 | .6518 | .14 | .14 |
| .982 | 1.5635 | .88 | 1.0485 | .62 | .6400 | .12 | .12 |
| .981 | 1.5496 | .87 | 1.0243 | .61 | .6283 | .10 | .10 |
| .980 | 1.5364 | .86 | 1.0018 | .60 | .6168 | .08 | .08 |
| .978 | 1.5118 | .85 | .9803 | .58 | .5940 | .06 | .06 |
| .976 | 1.4893 | .84 | .9597 | .56 | .5717 | .04 | .04 |
| .974 | 1.4685 | .83 | .9405 | .54 | .5496 | .02 | .02 |
| .972 | 1.4492 | .82 | .9218 | .52 | .5279 | .00 | .00 |
| .970 | 1.4312 | .81 | .9039 | .50 | .5065 | | |

In deriving these formulas for the water-surface curve, the coefficient was assumed to be a constant, but this is not always the case. In fact, in mo natural channels it will vary considerably, except in some carefully selecte reaches. Hence, average values are usually taken. Another assumption wa that the top width, T, of the section was so wide that the hydraulic radius, I

[6] Chesley J. Posey, in Engineering Hydraulics, by Hunter Rouse, John Wiley a Sons, Inc., New York.

ıld be assumed equal to two-thirds of the central depth, y. But again, in ie and deep sections, this will not be the case.

In a channel of parabolic form, if the top width of the water section, T, is ce the center depth, y, then the hydraulic mean depth, R, will be 0.45 y. If ratio T over y is increased to 4, R becomes 0.58 y, and for a ratio T/y of R will be 0.66 y. For greater values of this ratio, the hydraulic mean oth may be taken as two-thirds of the central depth of the parabola without ›reciable error. Let T/y equal r in the formula $R = \phi\ y$. Then for any par-›lic cross section with the ratio of top width to depth equal to or greater n two, the value of the coefficient of y can be computed by the formula:

$$\frac{1}{\phi} = 1.486 + 0.362\frac{1}{r} + 2.205\frac{1}{r^2} \quad \ldots\ldots\ldots\ldots\ldots \quad (7)$$

e average error in $\phi$ computed by Eq.7 in a dozen trials was less than $\frac{1}{4}$% h a maximum error of $\frac{3}{4}$% with r equal to 3. In practice, when working with a obtained from complete hydrographic surveys, Eq. 7 may become a mat- of academic interest only, since areas and wetted perimeters of channel ›ss sections are usually obtained from plots of the sections.

## SELECTING C AND n

If backwater curves are to be computed for widely different volumes of v on a given stream, as for low water, average and flood flows, and if the

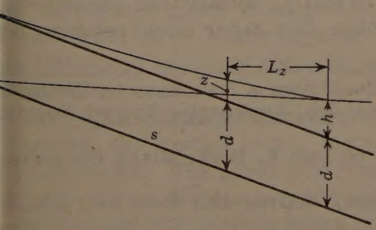


FIG. 1

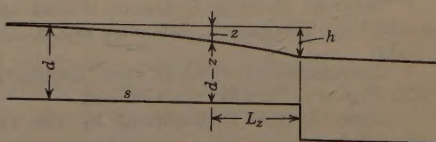


FIG. 2

əam bed is subject to scour, it is often difficult, if not impossible, to de- mine the characteristic depth (that is, an average or central depth for a tangular or parabolic section) during high stages of flow. Experience has wn that for every foot of rise during floods, there may be 2 ft or 3 ft of ır, or even more. In such cases, the exercise of one's best judgment based experience is necessary in the selection of coefficients and factors to be l in any formula. The selection of the coefficient of roughness, n, is a case oint. Experience indicates that the value of this important coefficient may ıge as much as from 0.030 to 0.025 or 0.020 with a large change in the hy-

draulic radius from low flow to flood conditions in a channel.[7,8] To quote or one authoritative statement,[9]

> "For example, the Manning coefficient of roughness changes a great deal in practically any channel when the variation in depth extends to a considerable range."

To avoid the necessity of making this adjustment, some engineers have us exponential formulas of various forms.[10] By this means, it was intended make the coefficient of roughness purely descriptive, as it was originally i tended to be. The more common practice at present, however, is to use t Manning formula and to adjust this coefficient as one's best judgment dictate

Additional material is available[11-18] for those who may wish to study son of the more recent papers and discussions on the subject of backwater curve

## CONCLUSIONS

While not new, Tolkmitt's tables are made available to American enginee in slightly enlarged form, and the use of the Manning velocity formula is re ommended with care in selecting the necessary coefficients.

## ACKNOWLEDGMENTS

The contributions of H. van der Veen have already been mentioned as pr viding the material which made it possible to offer this paper to the professio Grateful acknowledgment should also be given to Walter E. Jessup, F. ASC through whom a copy of the 1948 edition of Tolkmitt's book was loaned by t Engineering Societies Library for final checking of the equations and tabl shown herein. Mr. Posey kindly furnished information on the coefficient C adapted for use with the Manning formula. Finally, without the assistance R. D. Goodrich, Jr., in recomputing the tables, this paper could not have be prepared.

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7 "Diagram Relating Hydraulic Radius and Coefficient of Roughness for Ditches a Canals," by Clarence T. Johnston and R. D. Goodrich, Engineering Record, Novemb 4, 1911.

8 "Calculation of Flow in Open Channels," by Ivan E. Houk, Miami Conservan Tech. Reports, Part IV, pp. 136-143.

9 "Open-Channel Hydraulics," by Ven Te Chow, McGraw-Hill Book Co., Inc., N York, 1959.

10 "American Civil Engineers Handbook, H. and W. Formula," John Wiley and Son Inc., New York, Ed. 1930, p. 1346.

11 "Handbook of Hydraulics," by King, McGraw-Hill Book Co., Inc., New York, E 1954.

12 "Profile Curves for Open Channel Flow," by Dwite Gunder, Transactions, ASC Vol. 108, 1943, p. 481.

13 "A Direct Step Method for Computing Water Surface Profiles," by Arthur Eyra, Transactions, ASCE, Vol. 119, 1954.

14 "Backwater Functions by Numerical Integration," by Clint J. Keifer and Hen Hsien Chu, Transactions, ASCE, Vol. 120, 1955, p. 429.

15 "Backwater Effects of Open Channel Constrictions," by Hubert J. Tracy a Rolland W. Carter, Transactions, ASCE, Vol. 120, 1955, p. 993.

16 "Tranquil Flow Through Open Channel Constrictions," by Carl E. Kindsvater a Rolland W. Carter, Transactions, ASCE, Vol. 120, 1955, p. 955.

17 "Graphical Determination of Water Surface Profiles," by Francis F. Escoffie Proceedings, ASCE, No. 1114, HY Div., October, 1957.

18 "Open Channels with Nonuniform Discharge," by Wen-Hsiung Li, Transaction ASCE, Vol. 120, 1955, p. 255.

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# HOOD INLET FOR CLOSED CONDUIT SPILLWAYS[a]

By Fred W. Blaisdell,[1] F. ASCE

## SYNOPSIS

The hood inlet for closed conduit spillways is formed by cutting a circular ipe at an angle and placing the pipe so that the crown projects beyond the in- ert. This forms a hood over the pipe entrance.

The use of the hood inlet causes the pipe to fill with little or no submergence f the inlet crown. This filling occurs even though the pipe is laid on a slope hat is hydraulically steep. Smooth pipes having slopes greater than about 1.5%, nd corrugated pipes having slopes greater than about 8% are hydraulically teep. Therefore, many highway culverts and other drainage structures fall nto this steep category and will not fill unless the entrance receives special ttention.

The capacity and performance of the spillway for variations of the hood in- et length, the conduit slope, the wall thickness, and the approach conditions re described. The great effect of vortices on the spillway capacity is shown nd anti-vortex devices are developed. Scour in the vicinity of the hood inlet determined for various sizes of stone and equations for the scour-hole di- ensions are presented.

Note.—Discussion open until October 1, 1960. To extend the closing date one month, written request must be filed with the Executive Secretary, ASCE. This paper is part the copyrighted Journal of the Hydraulics Division, Proceedings of the American So- ety of Civil Engineers, Vol. 86, No. HY 5, May, 1960.

a Presented at the August 1957 ASCE Hydr. Div. Conference in Cambridge, Mass.

1 Proj. Supervisor, U. S. Dept. of Agric., Agric. Research Service, St. Anthony Falls ydr. Lab., Minneapolis, Minn.

## INTRODUCTION

Renewed interest in the hydraulic performance of culverts is evidenced by the number of studies reported since 1950. One reason for this is that the hydraulics of culverts and other types of closed conduit spillways is not as simple as was once thought. Another reason is because large numbers of these structures are being built each year and small savings or improvements in each structure result in large total savings.

Among the recent investigations is a study of the hood inlet which was discovered and briefly tested by Malcom H. Karr and Leslie A. Clayton, F. ASCE.[2] This appeared to be a simple, economical, and easily installed inlet that would be particularly useful to the United States Soil Conservation Service (SCS) engineers cooperating with soil conservation districts where the structures are installed by the landowner. The agricultural conservation, flood prevention, and watershed protection programs of the United States Department of Agriculture also would have use for this type of inlet. Accordingly, at the request of the SCS, a thorough study of the hood inlet was initiated by the Soil and Water Conservation Research Division of the Agricultural Research Service (ARS) at the St. Anthony Falls Hydraulic Laboratory, Minneapolis, Minn.

## PREVIOUS WORK

This review of previous work will be largely confined to closed conduit spillways as used by the SCS in its programs of soil and water conservation, flood detention, and watershed protection with the recognition that this restriction in scope necessarily excludes noteworthy pertinent information.

The earliest studies on closed conduit spillways were those conducted on drop inlet spillways under the direction of L. H. Kessler, F. ASCE, in 1933 and 1934.[3] A few field tests were made during the 1930's by plugging the drop inlet, waiting for runoff to fill the reservoir, and removing the plug.[4] The barrels of some of these spillways were on a slope steeper than that of the hydraulic friction grade line, yet the barrel flowed completely full—a phenomenon that previously had been thought impossible when the outlet was unsubmerged. As a result of these field tests and other field observations of full pipe flows, the SCS recognized a lack of knowledge of the hydraulics of closed conduit spillways and a need for additional laboratory investigations. They, in cooperation with the Minnesota Agricultural Experiment Station and the St. Anthony Falls Hydraulic Laboratory of the University of Minnesota, established a research staff at the St. Anthony Falls Hydraulic Laboratory in 1940.

Experiments on the hydraulics of closed conduit spillways were begun in 1941 and have been conducted, with interruptions, since that time by the SCS and the ARS to which the SCS research activities were transferred on January 1, 1954. During the early and middle 1940's, the theory of closed conduit spill-

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2 "Model Studies of Inlet Designs for Pipe Culverts on Steep Grades," by M. H. Karr and L. A. Clayton, Bulletin No. 35, Oregon State College, Corvallis, Oreg., Engrg. Experiment Sta., June, 1954.

3 "Experimental Investigation of the Hydraulics of Drop Inlets and Spillways for Erosion Control Structures," by L. H. Kessler, Bulletin No. 80, Univ. of Wisconsin, Madison, Wis., Engrg. Experiment Sta., 1934.

4 "Hydraulics of Closed Conduit Spillways, Part VIII, Miscellaneous Laboratory Tests," by Fred W. Blaisdell, St. Anthony Falls Hydr. Lab., Tech. Paper No. 19, Series B, Minneapolis, Minn., March, 1958, p. 20.

ay hydraulics was developed,[5] model-prototype relationships were tested and erified,[6] test methods were standardized, and methods of analysis were deeloped. Refinements were, and continue to be, made as experience indicates ieir desirability.

The earlier studies showed that the drop inlet must be five barrel diameters 5 D) high in order to insure full barrel flow when the barrel slope is steep and ie barrel entrance is square-edged.[6] For many installations this drop inlet eight is much greater than it is economically desirable to use. It has been ound possible to reduce the drop inlet height to two barrel diameters (2 D) if ie barrel entrance is enlarged.[7] But even this low drop inlet is too high under ome circumstances, so the search continued for a lower drop inlet or, preferbly, its complete elimination.

The investigation of the hood inlet by Karr and Clayton[2] indicated that it ould probably fill the need of the SCS for a simple closed conduit spillway inet which would insure that the barrel would fill even if it were laid on a steep lope. The studies reported in this paper can be considered as the developient of the hood inlet originated by Karr and Clayton.

As shown in Figs. 1 and 2, the hood inlet is formed by cutting a pipe at an ngle and laying the pipe so that the longer side is at the crown. This forms hood over the pipe entrance. The name was selected by ARS and SCS conerees as being descriptive of the inlet.

## THEORY

No hydraulic laws beyond those of elementary hydraulics are involved in omputing the flow through closed conduit spillways. However, their appliation to the hood inlet will be reviewed for the case of a spillway having a eep barrel slope.

The definition of conduit slope is in order. The slope is defined as steep if e slope of the conduit is steeper than the slope of the friction grade line. he pressure will ordinarily be sub-atmospheric throughout most of the conduit ngth if the outlet is unsubmerged. In contrast, a conduit with a flat slope has slope less than that of the friction grade line. The pressures in the conduit ill then be greater than atmospheric, except at the crown near the outlet when e outlet is unsubmerged. The friction grade line slope represents the rate loss of energy due to pipe friction. It is the hydraulic grade line slope when e losses are due to pipe friction alone, losses due to bends, enlargements, d other sources being excluded.

*Weir Flow.*—The conduit entrance acts as a weir if the conduit slope is steep d the head is less than the pipe diameter D. The head-discharge relationship expressed by the familiar equation

$$Q = c_w a\sqrt{2 g H} \quad \dots\dots\dots\dots \quad (1)$$

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[5] "Hydraulics of Closed Conduit Spillways, Part I, Theory and its Application," by ed W. Blaisdell, St. Anthony Falls Hydr. Lab., Tech. Paper No. 12, Series B, Minnpolis, Minn., January, 1952, Revised February, 1958, p. 11.

[6] "Hydraulics of Closed Conduit Spillways, Parts II through VII, Results of Tests Several Forms of the Spillway," by Fred W. Blaisdell, St. Anthony Falls Hydr. Lab., ch. Paper No. 18, Series B, Minneapolis, Minn., March, 1958, p. 24.

[7] Ibid., p. 36.

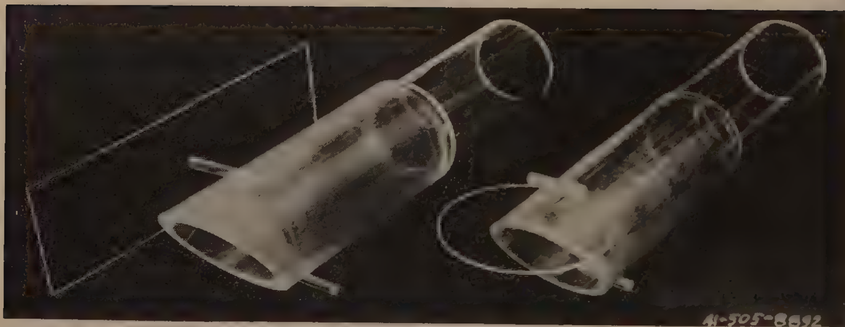


FIG. 1.—HOOD INLET AND ANTI-VORTEX DEVICE

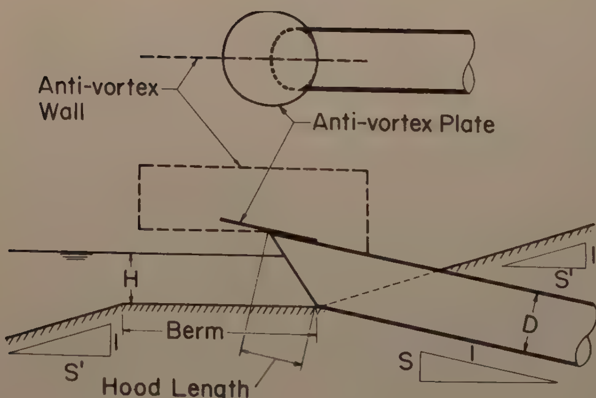



FIG. 2.—HOOD INLET DIMENSIONS

in which Q represents the discharge, $c_w$ is a dimensionless coefficient, a denotes the area of the barrel corresponding to the head H over the invert at the entrance, and g is the gravitational constant. Eq. 1 can be put into semi-dimensionless terms if both sides are divided by the product of the pipe area $A = \pi D^2/4$ and the square root of the pipe diameter D. After rearranging, this results in

$$\frac{Q}{D^{5/2}} = c_w \frac{\pi\sqrt{2\,g}}{4} \frac{a}{A} \sqrt{\frac{H}{D}} \quad \text{.............} \quad (2)$$

from which it can be seen that

$$\frac{\dfrac{Q}{D^{5/2}}}{\dfrac{a}{A}\sqrt{\dfrac{H}{D}}} = c_w \frac{\pi\sqrt{2\,g}}{4} = C_w = \text{constant} \quad \text{..........} \quad (3)$$

where $C_w$ has the dimension $ft^{1/2}$ per sec in English units.

If the barrel slope is flat, the head-discharge relationship will be determined by exit conditions, such as critical depth or the tailwater level, instead of the entrance acting as a weir.

*Pipe Flow.*—Pipe flow governs the head-discharge relationship when the conduit is full of water. The head between the headpool surface and the point at which the hydraulic grade line pierces the plane of the exit of a freely discharging pipe $H_t$ is consumed by entrance, friction, exit, and other losses such as bends, contractions, and expansions. All of these losses are related to the conduit velocity $V_p$ so that

$$H_t = \left(K_e + f\,\frac{l}{D} + K_o + \ldots\right) \frac{V_p{}^2}{2\,g} \quad \text{...........} \quad (4)$$

in which $K_e$ is the entrance loss coefficient, $K_o$ is the exit loss coefficient (assumed as 1.0 for these studies), f is the Weisbach friction factor, and l is the conduit length. Letting

$$H_t = H + Z - \beta\,D\,\cos\left(\sin^{-1} S\right) \quad \text{...........} \quad (5a)$$

and

$$Q = A\,V_p \quad \text{.....................} \quad (5b)$$

where Z is the elevation difference between the entrance and exit inverts, $\beta\,D\,\cos\,(\sin^{-1} S)$ is the vertical distance above the exit invert at which the hydraulic grade line pierces the plane of the exit, and S is the sine of the barrel slope. Solving for Q,

$$Q = A \sqrt{\frac{2\,g\left[H + Z - \beta\,D\cos\left(\sin^{-1} S\right)\right]}{K_e + f\,\frac{l}{D} + K_o + \ldots}} \quad \text{........} \quad (6)$$

*Correction for Viscous Resistance.*—The Froude model law was used to define the model-prototype relationship since the flow through the spillway is caused by gravitational forces. However, the resistance to full-pipe flow is due to viscous forces and these forces are of sufficient importance that the Reynolds

model law must be considered. Because these two laws are incompatible whe[n] the same fluid is used in model and prototype, corrections must be made fo[r] the viscous effects if the models are operated in accordance with the Froud[e] law. This correction is accomplished through the relation

$$\frac{h_f}{l} = \frac{f}{D} \frac{V_p^2}{2\,g} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (7$$

The slope of the hydraulic friction grade line for uniform established flo[w] in the barrel $h_f/l$ is a measure of the viscous effect. A length of straight barrel sufficient to permit a reliable evaluation of the friction slope therefore wa[s] made a part of the experimental apparatus. Multiplying both sides of Eq. 7 b[y] the conduit length l gives

$$h_f = f \frac{l}{D} \frac{V_p^2}{2\,g} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (8$$

the friction loss term in Eq. 4. The expression for $H_t$ in Eq. 4 is evaluated b[y] Eq. 5. Since $K_o$ is taken to be 1.0, $K_e$ in Eq. 4 can be evaluated. Thus, althoug[h] the model is operated in accordance with the Froude law, a correction for Reynolds law effects permits the evaluation of the entrance loss coefficient an[d] makes the entrance loss coefficient independent of the viscous resistance.

*Model-Prototype Similarity During Water-Air Flows.*—Quantitative similarity of water-air mixtures between model and prototype was not attempted. Fortunately, the presence of air during slug or mixture flows does not cause serious difficulty in predicting the quantitative performance of full-size structures. This has already been demonstrated for the case of the drop inlet close[d] conduit spillway.[6]

For the hood inlet, a steep conduit slope, and increasing discharge, it wil[l] be shown subsequently that air flow begins when the inlet seals and slugs form near the inlet. These slugs suck air through the inlet as they travel down the conduit. The amount of air is determined by the water seal at the inlet which the air has to penetrate and the suction head resulting from the traveling slugs. As the water flow increases, the headpool rises slightly and the seal at the inlet becomes thicker. At the same time, the slug length and the resulting suction become greater. The net effect is an increase in air flow. The air flow balances the slug air demand and the slug air demand is governed by the sum of the slug lengths. The air flow in the model will be different for the prototype than simple scaling would indicate, but the actual head over the inlet will be little, if any, different than that expected from Froude law relationships.

Maximum air flow will occur when the slugs are so frequent that they completely fill the conduit. This is also the beginning of water-air mixture flow. The head through the spillway for mixture flow is determined by the density of the water-air mixture and this effective head, less the friction head due to viscous resistance to flow, determines the amount of air drawn through the water seal at the inlet. Increasing water flow results in decreasing air flow until the air flow stops completely and ordinary pipe flow controls the head-discharge relationship at higher flows. As in the case of slug flow, the air flow rate in the form of a water-air mixture will, without doubt, be different in the prototype than would be predicted from model-prototype relationships. But the actual air flow is immaterial. The important consideration is that the head-discharge relationship and the differences between the model and its prototype will, in all likelihood, be of minor importance if the Froude law is used to scale the model test results to prototype quantities.

The head and the discharge at the beginning and end of air flow certainly can be predicted from the models with the usual reliability. For weir flow, air has no effect on the model-prototype relationship. For increasing water flow, the end of weir control is the beginning of slug (and air) flow and the head and discharge at this point can be definitely established and reliably projected to the prototype. Similarly, the beginning of full pipe flow is the end of air flow and the head and discharge for no air flow can be established with certainty and similarly projected to the prototype. Therefore, only during air flow is there any uncertainty at all in the model-prototype relationship. The prototype air flow can be expected to exceed the relative air flow in the model because of the greater actual prototype velocities. The relatively greater amount of air in the prototype will replace and thereby reduce the flow of water or, considering the headpool level, a greater head at the inlet will be required to achieve the same water flow. The hood inlet model tests show that a large change in rate of water flow causes only a small change in the headpool level during slug and mixture flows, a change in relative discharge of 40 producing only a unit relative change in head. Therefore, quantitative dissimilarity in air flow between a model and its prototype produces relatively much less dissimilarity in the headpool levels.

The fact: 1) that exact quantitative similarity according to the Froude law exists at the beginning and at the end of air flow, 2) that during air flow the increase in headpool level due to dissimilarity in air flow quantities over that predicted according to the Froude relationship is relatively small, and 3) that previous studies on different sized drop inlet closed conduit spillway models[6] have shown that the Froude law can be used to predict prototype water discharges during the flow of water-air mixtures, is sufficient justification for the earlier statement that air flow does not cause serious difficulty in predicting the quantitative performance of full size structures from tests made on their models.

## EXPERIMENTAL PROGRAM

A program of experiments was planned to develop the hood inlet so it could be used under a wide variety of field conditions. The experiments were designed to determine the optimum hood length for conduit slopes of 0% to 36%, the effects of conduit wall thickness and various approach conditions, the amount of scour to be expected in the vicinity of the inlet, and the type and size of anti-vortex device required. All tests were carried out with laboratory apparatus especially designed for the closed conduit spillway study.

## SPILLWAY PERFORMANCE

The test results will be presented in two sections. The first section will deal with the overall performance of the spillway. The second section will be devoted to the spillway capacity. In each section the various items investigated will be discussed in turn.

*Optimum Hood Length.*—The optimum length of the hood, measured parallel to the conduit centerline, was determined both quantitatively and by observation for six different conduit slopes. From five to eight different hood lengths were tested at each slope to evaluate the relationship between hood length and priming head. All tests were made with thin-wall, re-entrant entrances to simulate

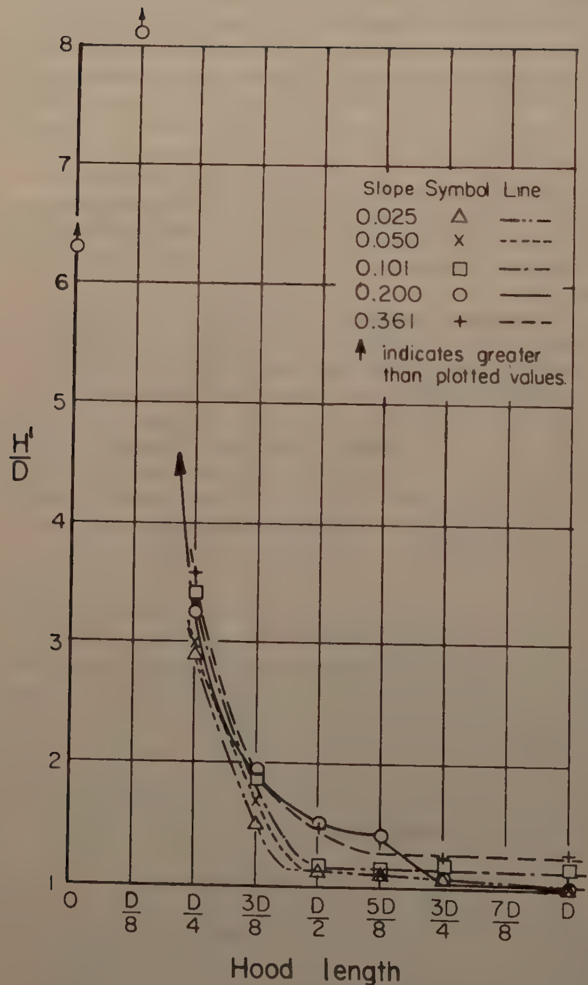



FIG. 3.—AVERAGE PRIMING HEAD AS FUNCTION OF HOOD LENGTH

what was thought to be the most severe operating conditions. The results of these tests are summarized in Fig. 3.

Priming ordinarily took place by sealing off the conduit near the inlet. This process is illustrated for a good hood inlet in Fig. 4. The headpool level is low in Fig. 4(a) so the inlet acts as a weir. Close observation shows that the water surface inside the inlet is at or very slightly above the headpool water level. The head in Fig. 4(b) is 0.91 D and "ears" are developing just inside the inlet. These ears are a local rise of the water surface that triggers the priming action. The head has increased to 0.98 D in Fig. 4(c) and the ears have grown until they almost meet. Twenty seconds later the head had increased to 0.99 D, the two ears had joined together, and the inlet had primed as is shown in Fig. 4(d). In another 60 sec the head had increased to 1.01 D, the inlet was completely full of water, and the pipe was full for a length of about 1 D as shown in Fig. 4(e). Filling the pipe creates additional head due to the slope of the pipe. This increases its capacity. Since there is insufficient seal over the inlet, air is sucked in as shown in Fig. 4(f). Further increases in flow and head result in the formation of slugs that travel through the pipe and suck in air as shown in Fig. 4(g). These slugs are hydraulic jumps that form near the inlet and travel through the conduit. The slugs increase in frequency until the conduit is full of a water-air mixture. As the water flow is increased, the air flow increases to a maximum and then decreases. Eventually the air flow stops, the conduit flows completely full of water as shown in Fig. 4(h), and the laws of pipe flow control the head-discharge relationship.

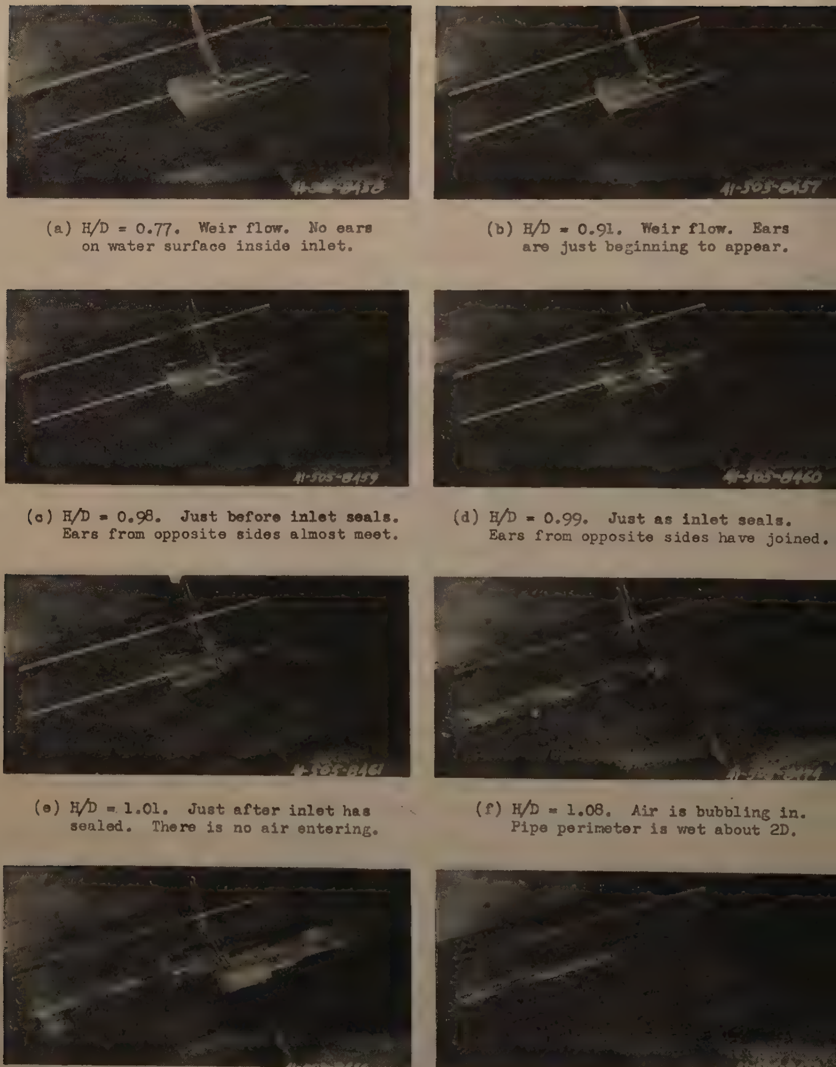
The development of the ears which determine the priming depends on the hood length. This is illustrated in Fig. 5 where the head for each photograph is approximately equal to the pipe diameter. The contracted jet expands and hits the inside wall at or below the horizontal centerline for the D/4 and the 3 D/8 hood lengths shown in Figs. 5(a) and 5(b). The point of impingement is higher for the D/2 hood length shown in Fig. 5(c) but the air-filled entrance contraction is readily apparent. There appears to be no air in the entrance contraction for the 5 D/8 hood length shown in Fig. 5(d). Also, ears are apparent. The ears have reached across the entrance crown and have sealed the inlet for the 3 D/4 and D hood lengths shown in Figs. 5(e) and 5(f).

The average priming head decreases as the hood length increases until a minimum is reached at a hood length of D/2 to 3 D/4. This is shown in Fig. 3. There seems to be no advantage in using a hood length longer than 3 D/4, and this length is recommended as the minimum satisfactory length. This minimum hood length could also have been closely determined by visual observations alone, as is illustrated in Fig. 5.

*Conduit Slope.*—The major effect of the slope of the conduit on the performance is in the additional head which becomes available as the slope and the total drop is increased. The effect of slope is large for the range of flows when air passes through the conduit and for full pipe flow but is small for weir control.

The slope affects the spillway performance for low discharges in the weir control range only for slopes so flat that the weir at the entrance is not the control section. This is shown in Fig. 6 where the only weir flow control data that deviate from the composite data is that for the zero conduit slope.

Full-flow discharge through the spillway increases with the conduit slope as is shown in Fig. 6. The conduit had the same length for all slopes, so the total drop through the spillway increased with the slope. The increase in total




(a) H/D = 0.77. Weir flow. No ears on water surface inside inlet.

(b) H/D = 0.91. Weir flow. Ears are just beginning to appear.

(c) H/D = 0.98. Just before inlet seals. Ears from opposite sides almost meet.

(d) H/D = 0.99. Just as inlet seals. Ears from opposite sides have joined.

(e) H/D = 1.01. Just after inlet has sealed. There is no air entering.

(f) H/D = 1.08. Air is bubbling in. Pipe perimeter is wet about 2D.

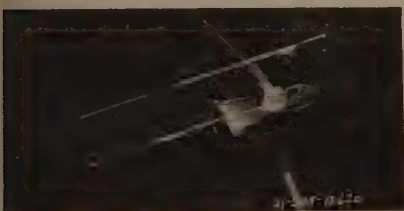
(g) H/D = 1.09. Hydraulic jump traveling through pipe and sucking air.

(h) H/D = 1.24. Pipe flow. No air enters inlet.

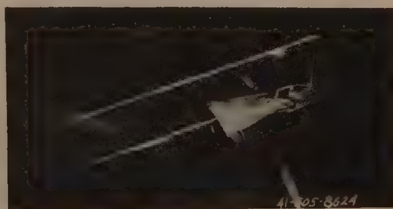
FIG. 4.—APPEARANCE OF HOOD INLET AS HEAD-POOL LEVEL RISES AND INLET PRIMES

head due to increasing the slope is actually what causes the increased flow. There was no change in the entrance loss coefficient with conduit slope.

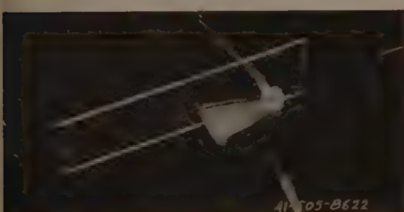
*Vortex Inhibitor.*—A device to control the formation of vortices at the entrance to closed conduit spillways is a necessity. The greatest need for the anti-vortex device is at low submergences of the inlet where a vortex may let air into the spillway; a vortex inhibitor is not needed for weir control at the entrance and is not essential when the inlet is deeply submerged.



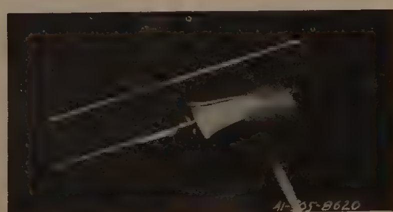
(a) Hood length = D/4.



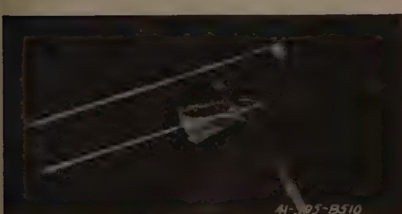
(b) Hood length 3D/8.



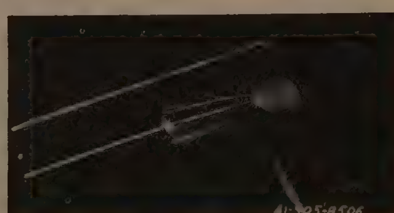
(c) Hood length = D/2.



(d) Hood length = 5D/8.



(e) Hood length = 3D/4.



(f) Hood length = D.

FIG. 5.—EFFECT OF HOOD LENGTH ON FLOW AT INLET FOR H/D = 1.0

The air entering through the vortex replaces the water and thereby reduces the discharge capacity of the spillway. The magnitude of the potential reduction in capacity as a result of vortices is apparent in Fig. 7 where data are plotted for two series of tests. The open circles and the curves represent data obtained with an anti-vortex wall. The anti-vortex wall was removed for the data identified by dots and run numbers. The data obtained for each run are connected to show the order in which they were obtained. Continuous curves could have been obtained if enough points had been read from the recorder chart. However, the data plotted are sufficient to illustrate the wide variation in the discharge as the vortex strength varies.

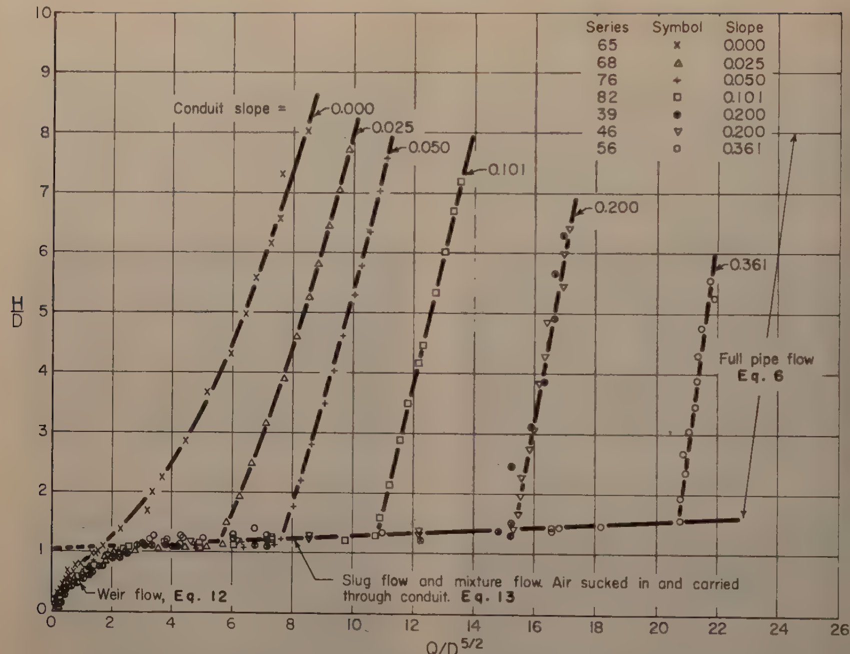



FIG. 6.—EFFECT OF CONDUIT SLOPE ON HEAD-DISCHARGE RELATIONSHIP. HOOD LENGTH IS 3 D/4

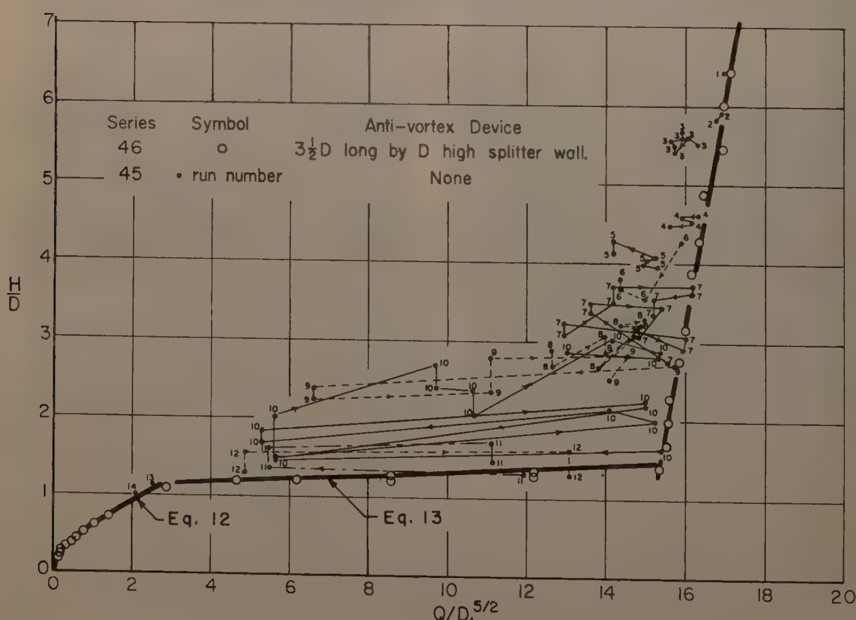



FIG. 7.—EFFECT OF PRESENCE OR ABSENCE OF AN ANTI-VORTEX DEVICE ON CAPACITY

The discharge is unpredictable either in magnitude or at any given time vhen vortices are present. In contrast, the completely reliable and predicta-ile head-discharge curve shown in Fig. 7 can be obtained if an anti-vortex de-rice is used. Since the capacity of the spillway of Fig. 7 was reduced to only ne-third its potential capacity by the elimination of the anti-vortex device, he importance of using some type of anti-vortex device is amply demonstrated.

The anti-vortex device used during the series plotted in Fig. 7 is a splitter vall mounted on the outside crown of the inlet shown in Fig. 8(a). This anti-ortex wall, or the alternate shown in Fig. 8(b), was used throughout most of he test program with uniformly satisfactory results. Although vortices some-imes formed, they admitted air to the conduit only briefly and did not cause a neasurable reduction in the spillway capacity.

Objections to the splitter type anti-vortex wall were voiced before the tests vere completed and a flat plate was suggested by Paul Jacobson, A. M. ASCE, J. S. Soil Conservation Service State Engineer for Iowa. Observational tests howed that the minimum size of anti-vortex plate is that dimensioned in Fig. (f).

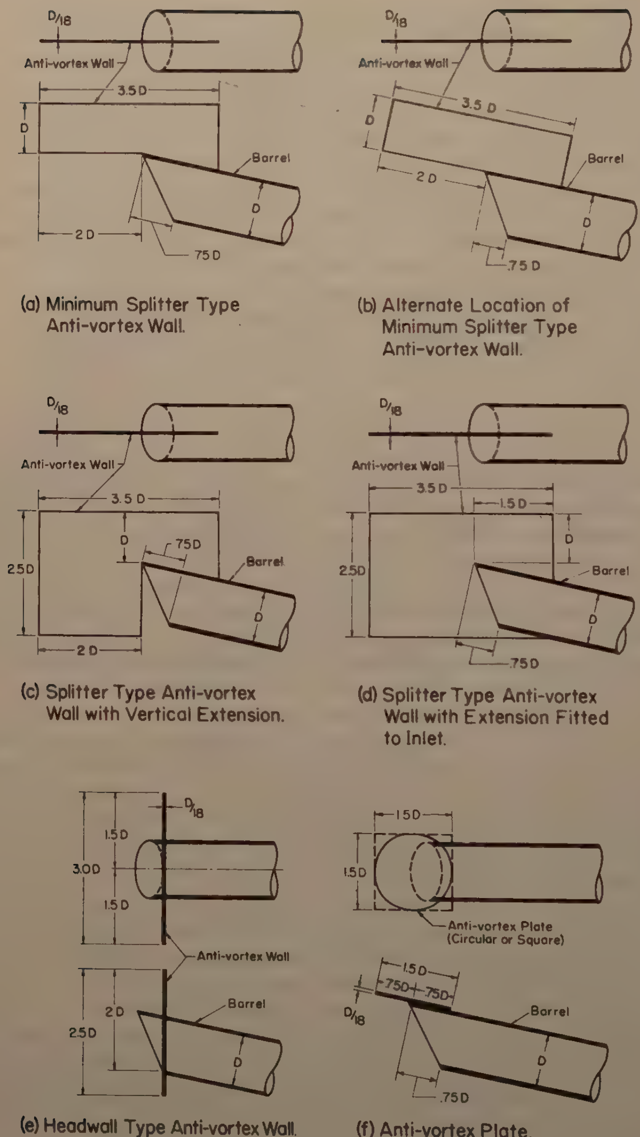
*Conduit Wall Thickness*.—A wide range of pipe wall thicknesses was tested vith the 3 D/4 hood inlet length and 20% conduit slope. The thickness of the hickest wall tested was 0.224 D. This is somewhat thicker than concrete pipe aving the greatest relative wall thickness as specified by the American Society or Testing Materials. The thinnest pipe wall tested was 0.015 D thick. At-empts were made to test wall thicknesses of 0.011 D and 0.006 D but the Lucite nodel was so thin (0.024 in. and 0.013 in., respectively) that the inlet broke efore data could be obtained. Even these thin walls are relatively thicker than he walls of metal pipe. Seventeen different wall thicknesses were tested.

The wall thickness had no apparent effect on the priming action or general nlet performance. Wall thickness did affect the loss at the entrance. The nagnitude of this effect is described subsequently.

*Approach Conditions*.—A limited number of different approach conditions vere tested. Most tests were conducted with the inlet re-entrant, that is pro-ecting into the headpool, since this was assumed to be the most severe test of nlet performance. The inlet performance was checked with the inlet invert ntersecting the sloping face of a dam and with a berm on the dam at the inlet nvert elevation. The face of the dam had a slope of 1 on 3.37 and the berm had width of 4.22 D. Two conditions of each dam shape were tested: For one ondition erosion was prevented by plastering the sand dam with neat cement nortar; for the other condition a scour hole was allowed to develop under maxi-num flow and the test was run with the scour hole.

The various approach conditions had no effect on the general performance f the spillway although the presence of the dam did reduce the entrance loss omewhat. The range of approach conditions tested is limited and deserves urther study. Nevertheless, unless the approach is greatly restricted, it ap-ears that approach conditions will not have a major effect on the general prim-ng characteristics or the type of flow in the spillway.

*Scour at Hood Inlet*.—The velocity at the sharp-edged inlet is theoretically nfinite. It is only logical to expect scour in the vicinity of the inlet if the ma-erial there is erodible. Fortunately, the velocity decreases rapidly with dis-ance away from the inlet and this limits the size of the scour hole to a toler-ble amount. A series of tests was performed to evaluate the dimensions of the ole that would be scoured in various sizes of noncohesive materials. A second




(a) Minimum Splitter Type Anti-vortex Wall.

(b) Alternate Location of Minimum Splitter Type Anti-vortex Wall.

(c) Splitter Type Anti-vortex Wall with Vertical Extension.

(d) Splitter Type Anti-vortex Wall with Extension Fitted to Inlet.

(e) Headwall Type Anti-vortex Wall.

(f) Anti-vortex Plate.

FIG. 8.—ANTI-VORTEX DEVICES

jective of these tests was to determine the size of riprap required to pre-ent scour. And a third objective was the determination of the height of the od inlet above the erodible material at which no scour would occur. These jectives were achieved by conducting tests at four different discharges on ght different sand sizes.

The surface radius of the scour hole r is plotted in Fig. 9. In the case of e berm, the radius is taken as one-third the sum of the scour hole width and stream length. When there is no berm, the radius is taken as one-half the idth of the scour hole. Variation in the results is to be expected. The re-arkable thing about Fig. 9 is that the variation is small enough so reasonable rves can be drawn through the data. The curve for $Q/D^{5/2} = 15$ is quite well fined. The same slope was used for the other values of $Q/D^{5/2}$. This slope s the data as well as any other slope. The slope of the curve is 1/5. By in-ection, the equation of the curves of Fig. 9 is

$$\frac{r}{D} = 0.15 + 0.04 \frac{Q}{D^{5/2}} \left(\frac{D}{d}\right)^{1/5} \qquad (9)$$

ere d is the actual bed material size.

The data can be used to determine the grain size, $d_i$, of the bed material ich will just be picked up by a given discharge. These data are plotted in g. 10 for those flows when only a few grains were picked up or for which ovement was classified as imminent. A surprisingly well-defined curve is tained which, for the data available, applies to grains of 3 mm or greater ameter. This curve has the equation

$$\frac{d_i}{D} = \frac{1}{20} \frac{Q}{D^{5/2}} - 0.075 \qquad (10)$$

The depths of the scour holes y are plotted in Fig. 11. Although there is nsiderable scatter to the data, this is not unexpected. The curves drawn in g. 11 were started at the zero depth of scour as given in Eq. 10. The slope the curves was determined by the better defined data obtained for the two allest grain sizes. The same slope was used for all curves. The curves of g. 11 define the depth of the scour hole and have the equation

$$\frac{y}{D} = \frac{1}{20} \frac{Q}{D^{5/2}} - \frac{d}{D} - 0.075 \qquad (11)$$

Eq. 11 gives fair results for bed material sizes of 3 mm or greater. The ecision of this equation is low but it may serve as an aid to judgment.

Eqs. 9, 10, and 11 represent the data. An ample safety factor should be ed for design purposes. For example, a stone 0.89 D in size was picked up en slightly disturbed. It passed through the pipe and smashed the inlet. The ne was heavy, having a specific gravity of 2.9. This points out the necessity using riprap of adequate size, of paving to prevent scour, or of allowing the our hole to develop. The minimum riprap size can be determined from . 10, the minimum area to be paved can be determined from Eq. 9, and the proximate depth of the scour hole can be determined from Eq. 11. Appropri-e safety factors should be applied to the computed values.

Eq. 11 can be used to determine the height of the inlet invert above the bed aterial if no scour is to occur. This height is computed for one bed material d flow. Tests made both with and without the berm confirmed the prediction at no scour would take place if the inlet is located a proper distance above e erodible bed.

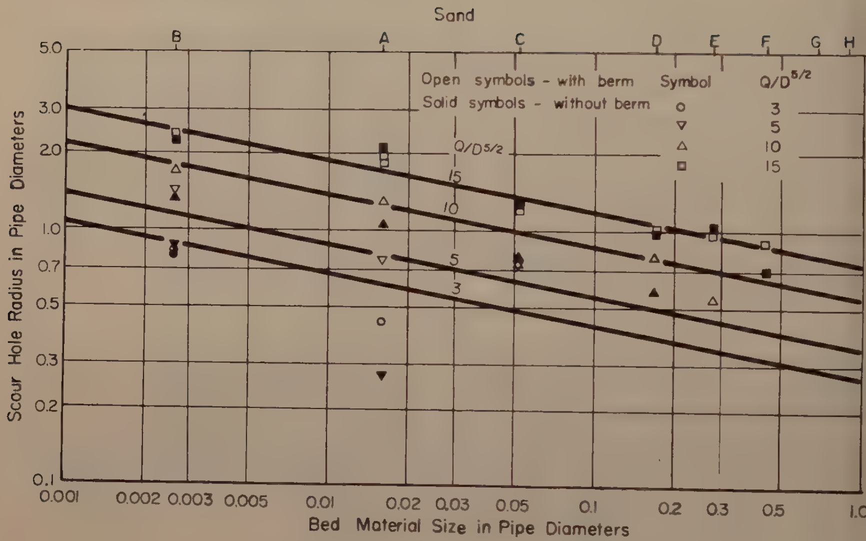



FIG. 9.—SCOUR HOLE RADIUS

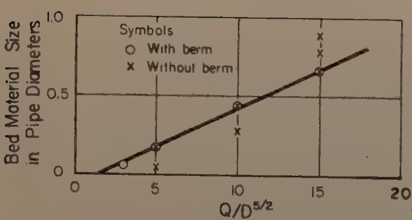



FIG. 10.—GRAIN SIZE IMMINENT MOVEMENT

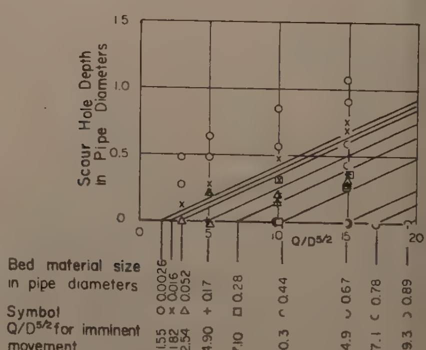



FIG. 11.—SCOUR HOLE DEPTH

The scour equations can be used to determine the minimum size of riprap required if the inlet crest is located at various distances above the surface of the riprap. If the distance below the inlet crest y and the discharge are substituted in Eq. 11, the size of riprap that will just be picked up can be determined. Also, the depth below the crest at which the available riprap will just not be picked up can be computed by inserting Q and d in Eq. 11.

Finally, smaller sizes of riprap can be used with increasing distance away from the inlet. These distances for various riprap sizes can be computed from Eq. 9.

These scour tests have defined reasonably well the relationships between the bed material size, the discharge, and the scour hole dimensions for various sizes of conduit. Minimum dimensions are given by Eqs. 9, 10, and 11, and it is essential that an adequate safety factor be applied if these equations are used for design purposes.

## SPILLWAY CAPACITY

At low discharges the control is the inlet acting as a weir. At intermediate discharges the control changes from weir to pipe and air is carried through the spillway with the water if the hood length is sufficient to cause the inlet to prime. At high discharges the control is as for full pipe flow. The spillway rating curve may be determined once the weir rating curve and the entrance loss coefficient are known.

*Capacity as a Weir.*—The capacity of the spillway with the entrance acting as a weir may be affected by the hood length, the conduit slope, the wall thickness, or the approach conditions. The effect of each of these items will be discussed in turn.

Hood Length.—The effect of hood length on the weir flow head-discharge relationship was determined for all conduit slopes, the data for all hood lengths being plotted on a single sheet for each conduit slope.

There is some scatter to the data but no trend that indicates any effect of hood length on the head-discharge relationship. This is somewhat surprising in view of the wide range of hood lengths, but no other conclusion is possible. These comments apply to the data for all conduit slopes tested.

Conduit Slope.—The conduit slope was found to have a small but significant effect on the weir flow head-discharge relationship. The data for one hood length and six conduit slopes appear in Fig. 12. The plots for other hood lengths show similar results.

The control for the 0.025 and greater conduit slopes is at the hood inlet crest and the data plot as a family of curves. However, the control for the zero conduit slope was critical depth at the conduit exit. The head-discharge relationship at the inlet is therefore dependent on the conduit friction loss in addition to the entrance losses. It will be different for each conduit length and roughness. Although the data for the zero slope are shown in Fig. 12, they should not be used for design purposes. The data can be used for design purposes when the conduit slope is sufficiently steep so that the control is always at the inlet.

Conduit Wall Thickness.—Head-discharge data covering the complete range of wall thicknesses tested were plotted and compared. The data scatter somewhat but the scatter is not greater than one can reasonably expect. There is no systematic trend in the scatter and a single curve can be drawn that adequately represents all wall thicknesses in spite of the wide range of thicknesses tested.

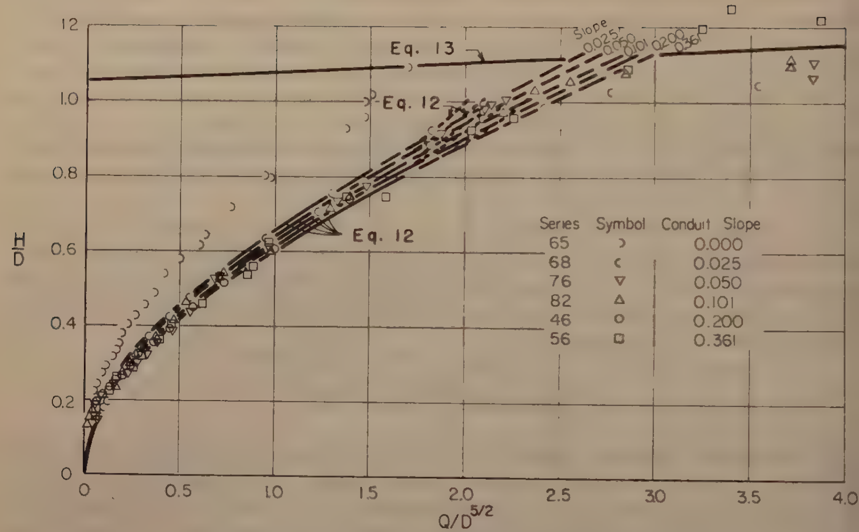



FIG. 12.—EFFECT OF CONDUIT SLOPE ON WEIR FLOW HEAD-DISCHARGE RELATIONSHIP. HOOD LENGTH IS 3 D/4

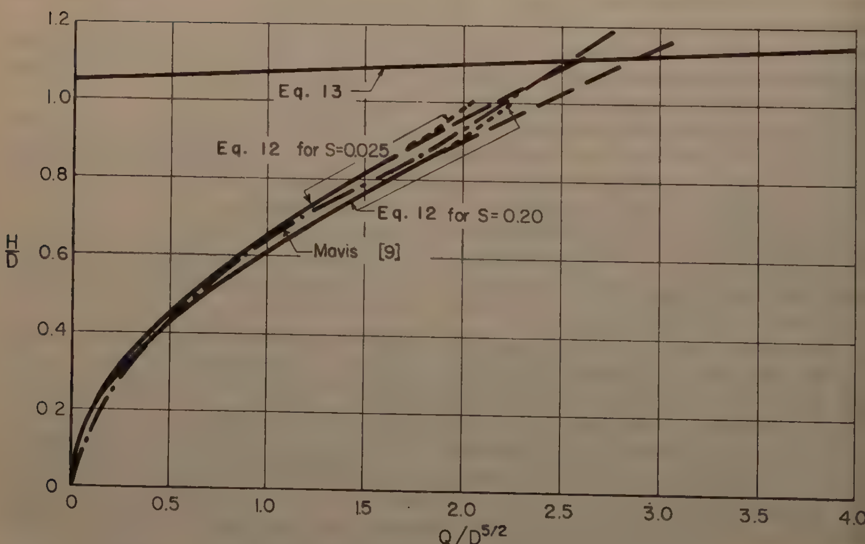



FIG. 13.—COMPARISON OF HOOD INLET WEIR FLOW HEAD-DISCHARGE EQUATIONS WITH MAVIS CURVE

The conclusion is that the wall thickness of the square-edged re-entrant ood inlet has no apparent effect on the weir flow head-discharge relationship.

Approach Conditions.—The number of different approach conditions tested too limited to permit any general statements regarding their effect on wier ow. The approach conditions tested include the sloping dam face and a berm the dam face at the crest elevation with both fixed and scoured surfaces.

The data show no effect of preventing scour or of allowing the scour hole develop. However, the presence of the dam appears to increase the discharge by approximately 10% over the condition for re-entrant inlets given by q. 12 but the present data are too few to justify general recommendations of creased flow for certain approach conditions.

Discharge Coefficient.—The weir flow coefficient $C_W$ in Eq. 3 was computed r the data shown in Fig. 12. Instead of a constant, it was found that a linear elationship existed between $\dfrac{\frac{Q}{D^{5/2}}}{\frac{a}{A}\sqrt{\frac{H}{D}}}$ and $\frac{H}{D}$. A different linear relationship as found for each conduit slope S. The relationship between $C_W$, S, and H/D r the square-edged hood inlet was found to be

$$C_W = 1.83\ S^{1/15} + 0.60\ \frac{H}{D} \quad \ldots\ldots\ldots\ldots$$

ich, when substituted in Eq. 2, gives

$$\frac{Q}{D^{5/2}} = \left(1.83\ S^{1/15} + 0.60\ \frac{H}{D}\right) \frac{a}{A} \sqrt{\frac{H}{D}} \quad \ldots\ldots\ldots (12)$$

The curve representing Eq. 12 is shown in Fig. 12. The agreement with the ta is seen to be excellent up to about 0.9 H/D. However, the equation starts deviate from the data at about H/D = 0.8 and the curves have been dotted ove this value. The dashed extensions of the head-discharge curve above D = 0.8 were drawn based on the shape of the curve given for the equation low H/D = 0.8 and it can be seen that they represent the data within the obined precision. Thus, Eq. 12 may be used to obtain the weir flow headscharge curve for square-edged inlets and conduits on steep slopes for heads tween H/D = 0 and H/D = 0.8. The curve may be extended to obtain the headscharge relationship for the higher heads.

It is shown later that weir flow is not the control for heads in excess of those ven by Eq. 13. The curve of Eq. 13 is shown in Fig. 12 to define this upper mit of weir control.

Comparison with Mavis' Curve.—The work of F. T. Mavis[8] on culvert hyaulics has been widely used and often quoted. A comparison of Eq. 12 with e weir flow head-discharge curve presented by Mavis for square-edged enances is therefore of interest. The principal difference between the inlets that Mavis used a plane headwall while here the inlets were re-entrant. so, Mavis' range of slopes was only from 0.007 to 0.0657 and he has indicat- no effect of slope. The comparison is made in Fig. 13. It is interesting to te that the agreement is generally good.

*Slug and Mixture Flow Capacity.*—The gently sloping line in Fig. 6 gives e head-discharge relationship for slug flow and water-air mixture flow through e conduit. It is also a limit curve giving heads and discharges below which e control will be weir flow and above which the control will be pipe flow.

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8 "The Hydraulics of Culverts," by F. T. Mavis, Bulletin No. 56, The Pennsylvania te College, Engrg. Experiment Sta., State College, Pa., 1943, Fig. 23.

The equation of the slug and mixture flow curve for the 3 D/4 hood leng may be approximated by

$$\frac{H}{D} = 1.05 + 0.025 \frac{Q}{D^{5/2}} \quad \ldots\ldots\ldots\ldots\ldots (1$$

*Extrance Loss for Full Conduit Flow.*—The entrance loss coefficient for fu flow is affected considerably by the hood length, the conduit wall thickness, t shape of the entering edge, and the approach conditions. The conduit slope ha no effect, and the effect of the various anti-vortex walls is not large.

Conduit Slope.—The average entrance loss coefficient $K_e$ for each seri shows no trend when plotted against the conduit slope S. The inlets were r entrant and the water had free access to all sides of the inlet for all condu slopes. Therefore, the finding that conduit slope has no effect on the entran loss is entirely reasonable.

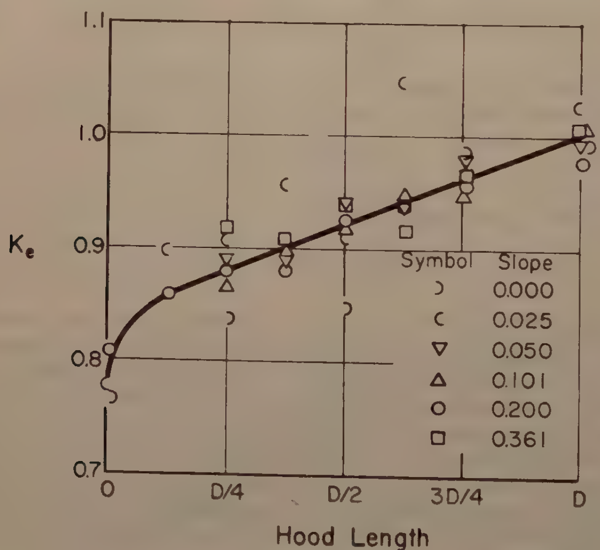


FIG. 14.—EFFECT OF HOOD LENGTH ON ENTRANCE LOSS CO-EFFICIENT. SQUARE-EDGED INLET, $t_p/D$ IS 0.015

Hood Length.—The effect of the hood length on the entrance loss coefficie is definite and considerable. The results of the tests are shown in Fig. 1 This significant effect for full flow contrasts with the lack of any such effe for weir flow. Fig. 14 demonstrates that there is a small but definite hydraul advantage in using the shortest hood length that will insure satisfactory hy draulic performance. This hood length is 3 D/4.

Conduit Wall Thickness.—The walls of the hood inlet may be classed as th or thick. The entrance loss coefficient $K_e$ is constant and the wall is class fied as thick when the ratio of the pipe wall thickness $t_p$ to the pipe diamete D is greater than 0.04. The loss coefficient is variable and the wall is class fied as thin when $t_p/D$ is less than 0.04. This is shown in Fig. 15. The dat plotted in Fig. 15 include data on walls as thin as it was possible to test. Thin ner walled inlets were built but they broke before data could be obtained. Th

ree low data points were obtained early in the test program. The reason why ey are low cannot be explained. The same inlets were tested again with the esults shown by points plotted directly above the low points.

The theoretical and experimental curves obtained by C. W. Harris[9] are own in Fig. 15 because they have the same shape as the curve obtained for

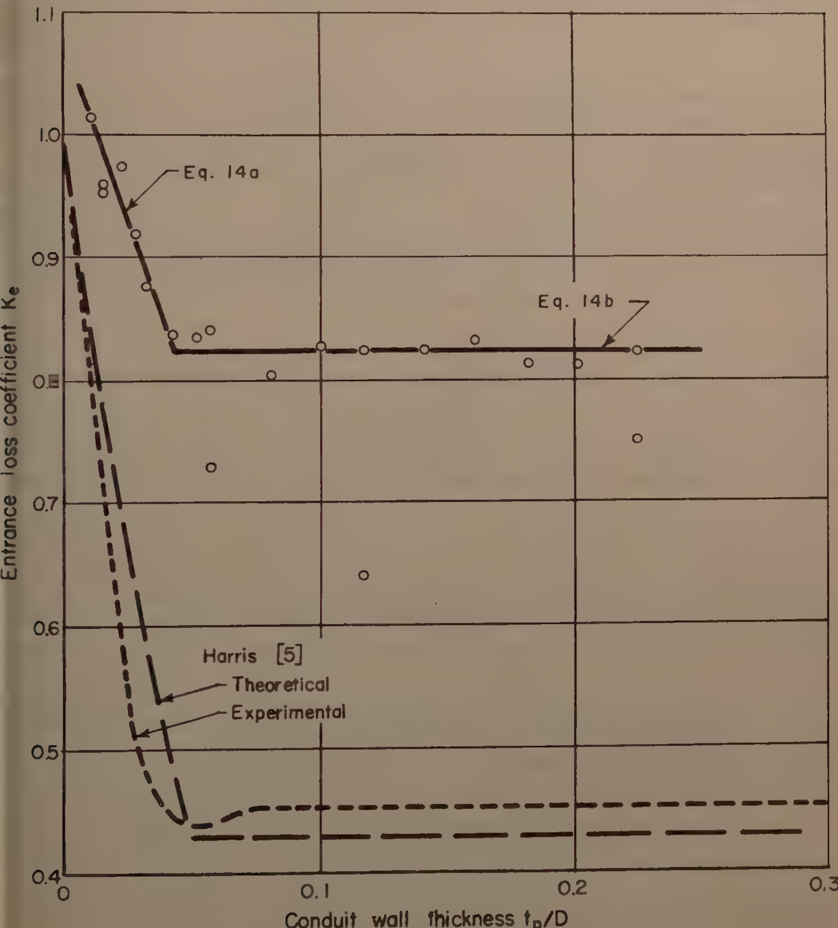


FIG. 15.—EFFECT OF CONDUIT WALL THICKNESS ON ENTRANCE LOSS COEFFICIENT. SQUARE-EDGED INLET. HOOD LENGTH IS 3D/4. CONDUIT SLOPE IS 0.20.

e hood inlet. The wall thickness obtained by Harris at which the coefficient eases to vary with wall thickness is seen to be the same as that obtained for e hood inlet.

9 "The Influence of Pipe Thickness on Re-entrant Intake Losses," by C. W. Harris, lletin No. 48, Univ. of Washington, Engrg. Experiment Sta., Seattle, Wash., November 1928.

The test results show, for the square-edged hood inlet having a hood leng of 3 D/4, that

$$K_e = 1.08 - 0.6 \frac{t_p}{D} \text{ for } \frac{t_p}{D} < 0.04 \quad \ldots\ldots\ldots\ (14$$

(thin-walled inlets), and that

$$K_e = 0.825 \text{ for } \frac{t_p}{D} > 0.04 \quad \ldots\ldots\ldots\ (14$$

(thick-walled inlets).

Anti-Vortex Device.—The various types of anti-vortex devices that we tested are shown in Fig. 8. Their effect on the entrance loss coefficient shown in Table 1.

TABLE 1.—EFFECT OF ANTI-VORTEX DEVICE ON ENTRANCE LOSS COEFFICIEN

| Series | Anti-Vortex Device | K |
|---|---|---|
| 39 | Fig. 8a | 0. |
| 46 | Fig. 8b | 0. |
| 47 | Fig. 8c | 0. |
| 48 | Fig. 8d | 1. |
| 49 | Fig. 8e | 0. |
| 106 | Fig. 8f | 0. |

TABLE 2.—EFFECT OF APPROACH ON ENTRANCE LOSS COEFFICIENT

| Series | Approach Condition | K |
|---|---|---|
| 51 | 1 on 3.37 dam face slope, fixed | 0. |
| 50 | 1 on 3.37 dam face slope, scour hole | 0. |
| 54 | 1 on 3.37 dam face slope, berm 4.22D wide, fixed | 0. |
| 53 | 1 on 3.37 dam face slope, berm 4.22D wide, scour hole | 0. |
| 46 | re-entrant | 0. |
| | Hood Length = 3 D/4 Conduit Slope = 0.20 t/D = 0.0156 Anti-Vortex Device = Fig. 8a | |

There is no difference in the behavior of the two splitter type anti-vorte walls shown in Figs. 8(a) and 8(b). They are so alike that no difference shoul be expected. Extending the anti-vortex wall down in front of the inlet and bring ing it closer to the inlet, as in Figs. 8(c) and 8(d), causes a small increase i the entrance loss coefficient. The headwall type anti-vortex wall shown i Fig. 8(e) apparently suppresses the contraction at the inlet and causes a con siderable decrease in the entrance loss coefficient. However, the efficienc of this wall in vortex control is somewhat less than the splitter walls. Th entrance loss coefficient for the anti-vortex plate shown in Fig. 8(f) is the low est obtained. This, then, is the best anti-vortex device, considering both it low loss coefficient and its vortex inhibiting ability.

Approach Conditions.—The use of a paved approach to the hood inlet results in a significant reduction in the entrance loss. This is shown in Table 2. The entrance loss coefficient is lowest if no berm is used.

The entrance loss is almost equal to that for the re-entrant inlet if the approach to the hood inlet is not paved and a scour hole is allowed to develop.

Such data as are presently available on different approach conditions show that a significant reduction in the entrance loss is possible if the approach is paved to prevent scour of the dam in the vicinity of the inlet.

## CONCLUSIONS AND RECOMMENDATIONS

Only the desirable weir and pipe controls exist for the hood inlet if the hood length is 3 D/4. The undesirable orifice control may exist for short hood lengths. The shortest hood which will insure full conduit flow at minimum headwater elevation is 3 D/4.

There is no influence of conduit slope on the spillway performance if the slope is steep.

A vortex inhibitor of some type is a necessity. The anti-vortex devices shown in Fig. 8 are recommended. However, the device shown in Fig. 8(e) is the least satisfactory.

The capacity of the spillway acting as a weir is given by Eq. 12; the capacity for flow of a mixture of air and water is given by Eq. 13; and the entrance loss coefficient for full pipe flow and square-edged entrances is given by Eqs. 14.

The composite head-discharge curve for a particular spillway having a good hood inlet can be drawn by plotting the weir flow curve, the slug and mixture flow curve, and the pipe flow curve. The extensions of these curves beyond their intersections with each other are imaginary so that the complete rating curve is formed by tracing, as the head and discharge increase, along the weir flow curve to its intersection with the slug and mixture flow curve, along this latter curve to its intersection with the pipe curve, and along the pipe curve for all higher heads and discharges.

The thickness of the conduit wall did not affect the hood inlet performance or its capacity as a weir. The effect of wall thickness on the entrance loss coefficient is given by Eqs. 14.

Although there was little variation in approach conditions, the approach had no influence on the spillway performance. The presence of the dam face does somewhat reduce the entrance loss coefficient.

## ACKNOWLEDGMENTS

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## APPENDIX NOTATION

The following symbols, adopted for use in the paper, conform essential with "American Standard Letter Symbols for Hydraulics" (ASA Z10.2-1942 prepared by a committee of the American Standards Association with Socie representation, and approved by the Association in 1942.

$a$ = wetted area of a circular pipe corresponding to a depth of flow of I

$A$ = area of a circular pipe = $\pi D^2/4$;

$A_o$ = orifice area taken at the plane of the entrance = $A \div \cos\theta$;

$c_o$ = orifice discharge coefficient;

$c_w$ = weir discharge;

$C_o$ = orifice discharge coefficient in feet-second units = $c_o \sqrt{2\,g}$;

$C_w$ = weir discharge coefficient in feet-second units = $c_w \pi \sqrt{2\,g/4}$;

$D$ = diameter of the pipe;

$f$ = Weisbach friction factor;

$g$ = acceleration due to gravity;

$h_f$ = friction head loss;

$h_n$ = local pressure deviation from the friction grade line;

$h_{vp}$ = velocity head in the pipe = $V_p{}^2/2\,g$;

$H$ = head over the invert at the entrance;

$H_o$ = head measured to the center of the entrance orifice = $H - (D/2)\Big[$co $(\theta - \sin^{-1} S)/\cos\theta\Big]$;

$H_t$ = total head from the water surface to the point at which the hydrauli grade line pierces the plane of the outlet = $H + Z - \beta D \cos(\sin^{-1} S)$

$K_e$ = entrance loss coefficient;

$K_o$ = exit loss coefficient;

$l$ = length of the conduit;

$Q$ = discharge through the conduit;

$\Delta Q$ = rate of change of headpool storage;

$S$ = sine of the slope of the barrel;

$t_p$ = pipe wall thickness;

$V_p$ = velocity in the pipe = $Q/A$;

$Z$ = difference in elevation between the conduit invert at the entrance and the exit;

$\beta$ = proportion of the pipe diameter above the invert at the pipe exit at which the hydraulic grade line pierces the plane of the exit; and

$\theta$ = angle between the face of the hood and a perpendicular to the conduit axis.

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# UNIFORM WATER CONVEYANCE CHANNELS IN ALLUVIAL MATERIAL[a]

By Daryl B. Simons,[1] M. ASCE and Maurice L. Albertson,[2] M. ASCE

## SYNOPSIS

Methods of designing uniform alluvial channels are developed and illustrat-d. Design theories which are recommended include: (1) a modification of the egime theory of India, and (2) a modification of the tractive-force theory. pecial emphasis is given to the modified regime theory. The results of this ivestigation are based on a field study of stable alluvial irrigation channels, nd other existing alluvial channel data which are applicable.

## INTRODUCTION

The design of silt stable or regime channels has been the object of consid-rable research during the past four decades. It is known that any design heory must recognize the effect of the many variables involved. Originally nly a single equation of the Chezy or Manning type was utilized. In recent ears, however, various investigators have concluded that width, depth, and lope are all variable in alluvial channels and that this implies mathematically he necessity of at least three design equations. The problem appears even ore complex when one recognizes that sediment transport also affects chan-el stability.

Recently, two schools of thought, one empirical, the other primarily theo-etical, have each evolved theories more capable of adequately predicting chan-

Note.—Discussion open until October 1, 1960. To extend the closing date one month, written request must be filed with the Executive Secretary, ASCE. This paper is part f the copyrighted Journal of the Hydraulics Division, Proceedings of the American So-iety of Civil Engineers, Vol. 86, No. HY 5, May, 1960.

[a] Presented at the 1958 ASCE Hydraulics Division Conference in Atlanta, Ga.

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nel behavior than any heretofore. As yet, however, these theories are incom-plete and should be subjected to further study and investigation in order broaden their scope and possibly reduce them into one general comprehensi and complete theory.

*Definition of a Stable Channel.*—An excellent definition of stable or regi channels was presented by E. W. Lane (14),[3] in 1953, as follows:

> A stable channel is an unlined earth channel for carrying water, the banks and bed of which are not scoured by the moving water, and in which objectionable deposits of sediment do not occur.

The foregoing definition does not exclude minor channel erosion or accr tion during the yearly cycle of flow. It does, however, require that these o posing effects should cancel on an annual basis.

*Application of Stable Channel Theory.*—Stable channel theory is widely a plicable to many phases of work in the fields of civil and agricultural enginee ing as follows:

Satisfactory design of irrigation systems requires an intimate knowledge channel design relationships. In fact, considering the current quality of the relations, considerable experience is necessary to bridge the existing gaps.

Improper design may well introduce instability, in artificial channels, such magnitude that it is not economically feasible to operate them.

Although canals constructed for the purpose of conveying drainage a wastewater from given areas might be considered under the foregoing hea ings, they have been subjected to such gross neglect that they warrant speci mention. Certainly lack of a suitable channel design theory, or at least lack application of it to problems falling in this group, has caused loss of conside able tillable acreage, many hydraulic structures, and the formation of unsigh ly scars on the earth's surface.

An adequate knowledge of the many variables influencing channel stabili and their interrelationships would make possible a more intelligent treatme of:

1. Bank-scour problems caused by increasing normal flow in channels.
2. Evaluation of the effect of slope changes, due to the use of cut-offs, channel stability.
3. Erosion problems upstream from and downstream from bridges a other types of hydraulic structures that constrict the channels.
4. Stabilization of non-regime channels by armor coating banks with mate rial more resistant to scour than the existing natural material and/or by in troducing structures to control slope.

It is of particular importance to be able to design stable power canals alluvial materials, since the quantity of sediment being transported to th power plant penstocks must be controlled—in fact, for the most part, elimi nated.

*Research.*—Based on the inadequacy of current design methods and lack understanding of the regime theory in the United States, a field study of stab canals was proposed in 1953, with the purpose in mind of attempting to clarif expand, and perhaps combine the foregoing theories. Specifically, the maj objectives of this research are as follows:

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[3] Numerals in parentheses, thus (1), refer to corresponding items in the Bibliograph

1. To investigate the validity and applicability of the regime theories, de-loped in India, to canals in the United States.
2. To investigate, expand, and possibly improve the tractive-force method stable channel design.
3. To relate the regime theories to the tractive-force theory, insofar as ssible.

In this paper, the results of the study of the applicability of the regime-type lationships to channel design are emphasized.

## REVIEW OF CURRENT DESIGN METHODS

Satisfactory design and construction of artificial channels in alluvial ma-rial is affected by many factors, some of which are extremely complex and nce are only vaguely comprehended. A brief history of the development of rrently used empirical rules and theories is presented herewith.

*Regime Theory.*—The regime theory of India was initiated by Kennedy, in 95, when he produced his classic empirical equation

$$V_0 = C\,D^m \qquad (1)$$

ere C and m were thought to be constants and were originally assigned values 0.84 and 0.64, respectively. Kennedy concluded that channels having veloci-es based on his formula would neither silt nor scour their beds. The equa-on was empirically formulated based on data collected on the Bari Doab ca-l system, in the Punjab.

According to Eq. 1 it is permissible to design a narrow deep channel or wide allow one to carry the same discharge. Actually, this is far from the truth the matter. As far as design is concerned the only time the Kennedy equa-on can possibly yield correct results is when shape is also properly selected, when dealing with very stable materials.

Other investigators attempting to prove or disprove Eq. 1 soon established at the constant C, and the exponent m, varied from system to system. In ite of this, the Kennedy equation, in its original form, can still be found in any recent engineering texts and it was applied extensively in design, as or-nally presented, until about 1930.

In 1919, E. S. Lindley introduced the following regime equations:

$$V = 0.95\,D^{0.57} \qquad (2)$$

$$V = 0.57\,B^{0.355} \qquad (3)$$

d

$$B = 3.80\,D^{1.61} \qquad (4)$$

ese equations were derived by correlating data obtained from 786 observa-ns in branch surveys of the Lower Chenab Canal. This was the first time

that bed width and depth were introduced as regime variables. In his reply the discussion on his paper he stated:

> The existence of these relations meant that the dimensions width, depth, and gradient of a channel to carry a given supply loaded with a given silt discharge were all fixed by nature, that is, uniquely determined.

The variables bed width, depth, and slope were observed. Velocities, howeve were not observed. They were computed by means of the Kutter and Che equations, the Kutter equation being:

$$C = \frac{41.65 + \frac{0.00281}{S} + \frac{1.811}{n}}{1 + \frac{n}{\sqrt{R}}\left(41.65 + \frac{0.00281}{S}\right)} \quad \text{. . . . . . . . . . . . . . . . (}$$

The magnitude of n in Eq. 5 was assumed constant at 0.0225. The Lindl equations, although never popular in the United States, were used extensive in India, until about 1935.

In 1927, Gerald Lacey was commissioned by the Governments of Engla and India to systematize all data that had been collected relative to stable cha nels. A summary of the results of the study were published in 1929 and 193 Lacey's original equations, as presented in the first paper, were:

$$V = 1.17 \sqrt{f\,R} \quad \text{. . . . . . . . . . . . . . . . . . . . . . . (}$$

$$A\,f = 3.8\,V^5 \quad \text{. . . . . . . . . . . . . . . . . . . . . . . . . (}$$

$$S = 0.000387\ f^{3/2}/Q^{1/9} \quad \text{. . . . . . . . . . . . . . . . (}$$

and

$$P = 2.668\,Q^{1/2} \quad \text{. . . . . . . . . . . . . . . . . . . . . . . (}$$

In 1936, N. K. Bose (12), and the staff of the Punjab Irrigation Research I stitute, presented the following formulas:

$$P = 2.8\,Q^{1/2} \quad \text{. . . . . . . . . . . . . . . . . . . . . . . . (1}$$

$$S \times 10 = 2.09\ d^{0.86}/Q^{0.21} \quad \text{. . . . . . . . . . . . . . (1}$$

and

$$R = 0.47\,Q^{1/3} \quad \text{. . . . . . . . . . . . . . . . . . . . . . . . (1}$$

Eqs. 10, 11, and 12 represent the results of several years of painstaking co lection and statistical analysis of the data. Note the similarity of Eqs. 9 a 10.

According to Lacey, f depends on size of sediment particles but is not af-
ted by concentration of the silt load being transported by the stream ($f = 3/4$
). In applying the Lacey theory, Q is known, f is estimated, and bed width,
pth, and slope are calculated.

In 1951 Thomas Blench presented his concept of the regime theory. His
lations are expressed in terms of W and D, where W is the mean width and
s the depth to an average line through the channel bottom such that the area
the cross section is WD. The design equations presented by Blench are:

$$W = \sqrt{\frac{b}{s}}\, Q^{1/2} \quad \text{(13)}$$

$$D = \sqrt{\frac{s}{b^2}}\, Q^{1/3} \quad \text{(14)}$$

$$S = b^{5/6}\, s^{1/12}\, Q^{-1/6}/2080\, r \quad \text{(15)}$$

which $r = \left(\frac{10^{-5}}{\gamma}\right)^{1/4}$, b = bed factor = $V^2/D$, and s = side factor = $V^3/W$.

At least three American engineers, C. R. Pettis (15) and L. B. Leopold and
Maddock, Jr. (16), have investigated the applicability of the regime theory
American rivers. The equations presented were of the same form as the
gime equations but did not have the same magnitude of constants and expon-
s. Hence, one might conclude that regime equations depend on the condi-
ns on which they are based and are valid only within the range of the ob-
rved data.

*Inadequacy of Regime Methods.*—The Lacey equations, and modifications of
em, have been very popular in India, but have never been used extensively
ewhere in the world. This group of equations undoubtedly provides as sound
basis for design as currently exists if they are used under circumstances
nilar to those from which they were obtained. The major disadvantages of
method are:

1. It has not been developed based on the wide variety of conditions encoun-
ed in practice;
2. It fails to recognize the important influence of sediment charge on design;
3. It involves factors that require a knowledge of the conditions upon which
formulas are based if they are to be applied successfully.

The regime equations presented by Blench modify the Lacey equations in
h a way that the effect of the side and bed of the channel can be evaluated
parately by means of the side factor and bed factor. This approach seems
ically more sound than averaging the two effects as the Lacey equations do
ce effect of bed and side conditions on flow are vastly different.

*Tractive-Force Method.*—The tractive-force design theory is formulated on
basis that stability of bank and bed material is a function of the ability of
bank and bed to resist erosion resulting from the drag force exerted on
m by the moving water.

This concept has been widely applied to the theory of sediment transpo both in the United States and in other countries but only to a limited extent connection with design of channels in alluvial material. Use of this meth for design has been suggested by Williams (14) and Armin Schoklitsch (2( The latter suggests that the relations given in Table 1 exist between type soils and permissible tractive force, and that these data are suitable for d sign purposes.

TABLE 1.—VARIATION OF PERMISSIBLE TRACTIVE FORCE WITH TYPE OF SOI

| Soil | Permissible tractive force, in pounds per foot |
|---|---|
| Loam | 0.062 |
| Sand | 0.102 |
| Stony and Loamy soil | 0.082 |
| Course gravel | 0.205 |
| Very compact soil | 0.256 |

The foregoing tractive-force concept was developed further under the lea ership of Lane (14), while with the United States Bureau of Reclamation (USBR The USBR procedure is based on the hypothesis that practical canal design a tractive-force problem—beyond that, the approach is new and theoretical.

Three distinct classes of instability have been defined by Lane as follows

1. Channels subjected to scour that do not silt;
2. Channels in which objectional deposition occurs but do not scour; and
3. Channels in which objectional scour and silting both occur.

Class 1 instability is the simplest of the three proposed, and fortunately, is also the one of primary importance, since most of the present and futu canal problems are and will be clear-water problems.

The recommended design procedure was developed by considering:

a. Distribution of tractive force over the channel periphery for differe side slopes with special emphasis on the magnitude of shear exerted on t sides as compared to the bed;

b. Relative stability of soil particles on the bed and on the sloping sides the channel; and

c. Magnitude of safe tractive force for different mean sizes and gradatio of non-cohesive materials.

The shear distribution was worked out mathematically for rectangular cha nels, and by membrane analogy and the method of finite differences for trap zoidal sections. It was found that maximum shear on the bed was approximat ly equal to $\gamma$ D S, and on the sides it was about 0.76 of this value.

Various theories have been developed making it possible to estimate magn tude of tractive force resulting from the movement of a fluid with respect to fixed boundary with which it is in contact as follows:

$$\tau = \gamma \, D \, S \ldots\ldots\ldots\ldots\ldots\ldots (1$$

ı which $\tau$ is the tractive force exerted on the bed at a point where the water epth is D. The derivation of Eq. 16 is based on equilibrium of forces tangent ɔ the channel boundary;

$$\tau = \gamma \, R \, S \quad \ldots\ldots\ldots\ldots \quad (17)$$

ı which $\tau$ is the average tractive force exerted on the channel periphery (Eq. 7 is derived using the same procedure as that for Eq. 16);

$$\tau = e\left(\frac{V_2 - V_1}{5.75 \log \frac{Y_2}{Y_1}}\right)^2 \quad \ldots\ldots\ldots\ldots \quad (18)$$

ı which $\tau$ is defined as the tractive force on the channel bed directly beneath ıe vertical in which $V_1$, $V_2$, $Y_1$, and $Y_2$ are measured (The derivation of Eq. 3 involves use of shear theory and the Karman logarithmic velocity distribu-on law); and $\tau$ can be computed by utilizing lines of equal velocity in the cross ection (isovels) to establish lines of zero momentum exchange. Then, com-ıting the tractive force on the bed between two zero momentum lines (lines rawn at right angles to the isovels) by considering the weight of water above ıe section of bed in question that is confined between the two zero momentum nes.

*Inadequacy of the Tractive-Force Method.*—The tractive-force concept is ısically sound insofar as it has been developed. That is, for the design of ıannels for the conveyance of essentially clear water in coarse non-cohesive ıaterials. However, its application is qualitative for the:

1. Design of channels in fine non-cohesive soils (sand range and finer);
2. Design of channels in cohesive materials; and
3. Design of channels that are required to transport appreciable (in excess : 500 ppm) sediment load.

## FACTORS INFLUENCING STABILITY OF OPEN CHANNELS

A detailed study of existing design methods immediately verifies the com-exities of stable channel theory. To cover adequately each existing design ıse, any suitable theory must include the effect of all of the pertinent vari-ɔles. To date, no theory has been conceived capable of adequately consider-g all of them and their influence on channel behavior. A list of the major ıriables involved follows:

1. Discharge condition;
   a. Steady discharge;
   b. Variation of discharge with crop needs;
   c. Periodic operation;
   d. Design discharge exceeded;
2. Slope;
3. Shape of channel;
4. Boundary material:

a. Cohesive materials;
b. Non-cohesive materials, sand range;
c. Non-cohesive materials, larger than sand;

5. Water temperature;
6. Magnitude and range in size of total sediment load;
7. Berms;
   a. Deposition of wash load;
   b. Shape of channels possessing berms;
   c. The influence of berm formation on bank vegetation;
8. Bank stabilization;
9. Secondary circulation or transverse flow;
10. Seepage inflow or outflow;
11. Effect of wind action on stability of channels:
    a. Estimate of tractive shear exerted on the bed of a channel due t waves;
    b. Effect of percolation caused by waves on the stability of a permeabl bed;
    c. Effect of waves on bank stability; and
    d. Wind effect during the non-operating season.

## FIELD DATA

The data upon which this paper is based were obtained from canal studie in India [Punjab (18) and Sind (11)] and the United States [Imperial Valley (7) San Luis Valley (14), and canals in Wyoming, Colorado, and Nebraska (2) an (22)]. The study of irrigation canals in Wyoming, Colorado, and Nebraska wa jointly sponsored by the Corps of Engineers, the United States Geological Sur vey (USGS), the USBR, Colorado State University, and University of Wyoming Field data from selected canal reaches in these states were taken by D. B Simons and D. L. Bender during the summers of 1953 and 1954 (22), hereafte referred to as Simons and Bender data.

The data taken in the field from the selected straight stable canal reache included:

(1). Magnitude of discharge;
(2). Velocity distribution;
(3). Slope of water surface;
(4). Shape of canal cross section;
(5). Suspended sediment distribution;
(6). Total sediment load, whenever possible;
(7). Samples of bed material and side material;
(8). Armor coat samples;
(9). General condition of the bed;
(10). Temperature of the water; and
(11). Photographs.

A total of 24 reaches were investigated. Complete information on eac reach is presented in Tables 2 and 3.

*USBR Data.*—Because of the limited amount of data on tractive force fo coarse non-cohesive material, the USBR (14) investigated canals constructe in this type of material in the San Luis Valley of Colorado. The canals wer

ocated on an alluvial fan. The size of the natural material varied considerbly, decreasing in size from the apex outward. The canals constructed in the one were stable, straight, and of uniform cross section. Fifteen reaches of anals were investigated. Values of Q varied from 17 cfs to 1,500 cfs and lopes from $0.79 \times 10^{-3}$ to $0.97 \times 10^{-2}$.

The primary purpose of using these data is to increase the range of condiions considered and to establish more points for the correlation phases of is study.

*India Canal Data.*—A rather thorough study of the available literature on egime theory, its conception, and evolution was undertaken. During the course f this investigation, considerable information was found on the canal systems f India which was used to help establish and develop the regime theory. Two roups of these data were sufficiently complete and pertinent to warrant incluion for use in the theoretical analysis.

The first group of the data is for forty-two stable Punjab canals (reference 18), pages 60 through 64). Their capacity varies from 5 cfs to 9,000 cfs. lopes are on the order of $0.12 \times 10^{-3}$ to $0.34 \times 10^{-3}$, and the average diameter f the bed material is approximately 0.43 mm.

An insight to the magnitude of sediment load carried by the Punjab canals as obtained (reference (17), page 87). The mean silt intensity for the chanels listed is 0.238 g per l or 238 ppm. This value is within the order of magitude of sediment concentration in the canals measured by Simons and Bender. onsidering the small difference in magnitude of sediment load for these two roups of canals, it is anticipated that they will behave as a single group exept possibly where major differences in bed and bank conditions exist.

The second group of canal data was found in statement I (reference (10), ages 70 and 71) and statement II (reference (11), pages 74 and 75). These ata were collected from twenty-eight different reaches of thirteen Sind canals, nd according to the foregoing references are stable. Their capacity ranges rom 311 cfs to 9,057 cfs, slopes vary between $0.0592 \times 10^{-3}$ and $0.0995 \times 10^{-3}$ nd mean size of bed material is within the limits 0.0346 mm to 0.1642 mm.

The Sind canals seem to carry a larger amount of sediment than those of e Punjab, at least during part of each year. This is verified by studying the ata (12). In accordance with observations 3 and 4, taken in 1934, the magniide of the suspended sediment load ranges from 3.59 g per l down to 0.156 g er l or from 3,590 ppm down to 156 ppm. The silt in the Sind canals has a maller mean diameter than that found in the Punjab canals. With the somehat larger sediment load, it is anticipated that these canals will behave difrently from the Punjab canals and the canals studied by Simons and Bender nless sufficient wash load occurs in the Sind group to automatically compenate for the difference in conditions.

*Imperial Valley Canal Data.*—Data from four irrigation canals in the Imerial Valley canal systems were obtained from a technical bulletin by Samuel ortier and H. F. Blaney (7) and a report by B. C. Raju (19). These data are ique in that their sediment concentrations are relatively high, ranging from 500 ppm to 8,000 ppm (most of this concentration must be wash load), and eir bed and bank conditions are similar to those found in the Punjab canals, e Sind canals, and the canals investigated by the writer.

*Water Temperature Data.*—An average effective water temperature for the venty-four canal reaches investigated by the writer has been worked out for ach canal. These data are given in Table 3.

TABLE 2.—GENERAL INFORMATION ON CAN

| No. | Name | Location | Maximum Sustained Discharge, in Cubic Feet per Second | Discharge, in Cubic Feet per Second | Exten Banl Vegeta |
|---|---|---|---|---|---|
| 1 | Bijou, 53 | West of Ft. Morgan, Colo. T3N, R59W, S10 | 190 | 177 | Moder |
| 2 | Farmers | 5 miles east of Mitchel, Neb. T23N, R | 950 | 773 | Light |
| 3 | Ft. Laramie I | West of Torrington, Wyo. Mile 31.65 | 1125 | 1031 | Light |
| 4 | Ft. Laramie II | South of Lyman, Neb. Below mile 91.2 | 475 | 445 | Light |
| 5 | Ft. Laramie III | South of Lyman, Neb. at mile 87.18 | 530 | 510 | Light |
| 6 | Ft. Laramie IV | Southwest of Torrington, Wyo. at mile 38.5 | 1030 | 950 | Light |
| 7 | Ft. Morgan I | West of Ft. Morgan, Colo. T3N, R58W, S1 | 137 | 137 | Moder |
| 8 | Ft. Morgan II | 7 miles west of Ft. Morgan, Colo. T4N, R58W, S18 and 19 | 190 | 191 | Light |
| 9 | Ft. Morgan III | 1 mile west of Ft. Morgan, Colo. T4N, R58W, S36, R55W, S30 | 160 | 160 | Light |
| 10 | Ft. Morgan IV | West of Ft. Morgan T4N, R58W, NW1/4 sec 28 | 170 | 171 | Light |
| 11 | Ft. Morgan V | West of Ft. Morgan T4N, R59W, NE1/4 S13 | 200 | 198 | Light |
| 12 | Garland I | 10 miles west of Powell, Wyo. S of H.W. 14, T54N, R100W, S17 | 880 | 883 | Light |
| 13 | Garland II | 4.5 miles W of Powell, Wyo. Parallel to H.W. 14, T55N, R99W | 750 | 751 | Light |
| 14 | Interstate | North of Morril, Neb. at mile 64.7 | 1200 | 1039 | Light |
| 15 | Larimer Weld | Immediately west of intersection of the Canal and H.W. 87 west of Ft. Collins, Colo. | 600 | 600 | Light |
| 16 | Lucerne I | 16 miles west of Torrington, Wyo. Barnes Siding, T26N, R63W, S32 | 55 | 55 | Moder |
| 17 | Lucerne II | 1 mile west of Barnes Siding Parallel to H.W. 26, T26N, R63W, S30 | 60 | 56 | Moder |
| 18 | North Platte Ditch | 4.5 miles west of Torrington, Wyo. Parallel to H.W. 26, T25N, R62W, S36 | 45 | 43 | Heavy |
| 19 | Bijou, 54 | West of Ft. Morgan, Colo. T3N, R59W, S10 | 190 | 199 | Moder |
| 20 | CNPP and ID | 0.7 miles N of Funk, Neb. below mile 33.8 | 240 | 236 | Heavy |
| 21 | Lat A29.1 | 0.5 miles NE of Holdrege, Neb. at mile 0 | 110 | 113 | Heavy |
| 22 | Cozad | South of Gothenburg, Neb. T11N, R25W, S17 | 240 | 227 | Moder |
| 23 | Dawson | South of Cozad, Neb. T10N, R23W, S7 | 360 | 363 | Light |
| 24 | Taylor Ord | West of Taylor, Neb. at mile 3.4 | 180 | 181 | Heavy |

[a] Stable, except where otherwise noted. [b] None in all cases.

VESTIGATED BY SIMONS AND BENDER

| Bank Material | Stability[a] | Maintenance Required[b] | Bern Formation | Method of Operation During Season | No. |
|---|---|---|---|---|---|
| derately cohesive | | | W. bank | Intermittent | 1 |
| derately cohesive | | | None | Continuous | 2 |
| hesive | | | Very little | Continuous | 3 |
| hesive | | | Very little | Continuous | 4 |
| hesive | | | Very little | Continuous | 5 |
| hesive | | | None | Continuous | 6 |
| hesive | | | Very little | Slightly intermittent | 7 |
| n-cohesive | | | None | Slightly intermittent | 8 |
| n-cohesive | | | Very little | Slightly intermittent | 9 |
| n-cohesive | | | None | Slightly intermittent | 10 |
| n-cohesive | | | None | Slightly intermittent | 11 |
| avel | | | None | Continuous | 12 |
| avel | | | None | Continuous | 13 |
| derately cohesive | | | None | Continuous | 14 |
| derately cohesive | Marginal | | None | Intermittent | 15 |
| derately cohesive | | | Very little | Continuous | 16 |
| derately cohesive | | | Very little | Continuous | 17 |
| derately cohesive | | | Very little | Continuous | 18 |
| derately cohesive | | | W. bank | Intermittent | 19 |
| hesive | | | Very little | Continuous | 20 |
| hesive | | | Very little | Continuous | 21 |
| n-cohesive | | | None | Continuous | 22 |
| n-cohesive | Marginal | | None | Continuous | 23 |
| n-cohesive | | | Very little | Continuous | 24 |

TABLE 3.—SUMMARY OF

| No. | Q | A | V | S x $10^3$ | R | P | D | $W_T$ | $W = \frac{A}{D}$ | Water Temperature $F^o$ |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 117.0 | 73.0 | 2.42 | 0.330 | 2.37 | 30.8 | 2.92 | 27.0 | 25.0 | 79.0 |
| 2 | 773.0 | 311.0 | 2.48 | 0.130 | 4.78 | 65.0 | 5.85 | 62.0 | 53.2 | 69.0 |
| 3 | 1,031.0 | 602.9 | 1.71 | 0.058 | 6.70 | 90.0 | 8.29 | 80.0 | 72.8 | 72.0 |
| 4 | 445.0 | 231.6 | 1.92 | 0.063 | 4.66 | 49.6 | 6.01 | 44.0 | 38.5 | 73.8 |
| 5 | 510.0 | 242.6 | 2.10 | 0.074 | 4.63 | 52.2 | 5.81 | 47.0 | 41.8 | 73.0 |
| 6 | 950.0 | 531.3 | 1.79 | 0.058 | 6.36 | 83.5 | 7.66 | 86.0 | 69.4 | 74.0 |
| 7 | 146.3 | 107.5 | 1.36 | 0.135 | 2.83 | 38.0 | 3.51 | 34.0 | 30.6 | 77.0 |
| 8 | 191.0 | 120.7 | 1.58 | 0.290 | 2.20 | 54.8 | 2.63 | 53.0 | 45.9 | 77.0 |
| 9 | 160.0 | 114.8 | 1.39 | 0.190 | 2.47 | 46.4 | 2.93 | 44.0 | 39.2 | 77.0 |
| 10 | 170.8 | 102.1 | 1.67 | 0.237 | 2.46 | 41.5 | 2.91 | 39.5 | 35.2 | 79.0 |
| 11 | 198.0 | 117.7 | 1.68 | 0.268 | 2.43 | 48.4 | 2.81 | 46.0 | 41.9 | 79.0 |
| 12 | 883.0 | 391.1 | 2.26 | 0.181 | 6.01 | 65.0 | 7.88 | 60.0 | 49.6 | 62.0 |
| 13 | 751.0 | 292.3 | 2.57 | 0.166 | 4.64 | 63.2 | 5.73 | 60.0 | 51.1 | 61.5 |
| 14 | 1,039.0 | 413.8 | 2.51 | 0.120 | 6.23 | 66.4 | 8.50 | 61.0 | 48.6 | 71.0 |
| 15 | 600.0 | 234.7 | 2.55 | 0.369 | 4.10 | 57.4 | 4.90 | 54.0 | 48.0 | 63.0 |
| 16 | 55.0 | 29.9 | 1.84 | 0.253 | 1.82 | 16.5 | 2.61 | 14.0 | 11.45 | 69.0 |
| 17 | 56.0 | 31.5 | 1.78 | 0.387 | 1.90 | 16.6 | 3.01 | 13.0 | 10.47 | 71.0 |
| 18 | 43.0 | 27.6 | 1.56 | 0.294 | 1.82 | 15.2 | 2.64 | 12.5 | 10.47 | 70.0 |
| 19 | 198.6 | 82.3 | 2.42 | 0.302 | 2.61 | 31.5 | 3.31 | 27.5 | 24.90 | 73.0 |
| 20 | 236.0 | 119.2 | 1.98 | 0.114 | 3.57 | 33.5 | 5.25 | 30.0 | 22.70 | 78.0 |
| 21 | 113.0 | 56.3 | 2.01 | 0.110 | 2.58 | 21.8 | 4.33 | 20.0 | 13.00 | 79.0 |
| 22 | 226.9 | 137.0 | 1.65 | 0.218 | 2.68 | 51.2 | 3.33 | 49.0 | 41.20 | 82.0 |
| 23 | 363.3 | 191.5 | 1.90 | 0.388 | 2.63 | 72.8 | 2.95 | 69.0 | 65.00 | 83.0 |
| 24 | 180.6 | 97.0 | 1.86 | 0.216 | 2.85 | 34.0 | 3.67 | 29.5 | 26.40 | 82.0 |

[a] Based on the analysis of one large sample.

The temperature of the San Luis Valley canals is not known, but, based on climatological conditions, an effective temperature of 65 F has been assumed for each of the fifteen reaches reported.

No specific information on temperature of individual canals in the Punjab or the Sind was found. It has been pointed out (reference (6) page 55), however, that temperature variation in the Punjab canals ranges from 9 C to 28 C, and that 20 C is a good average for the entire year. In addition, Blench (3) reports that the climate of the Punjab is similar to that of desert Arizona and that water temperatures vary from 50 F to 85 F. Based on the foregoing information, a base temperature of 70 F was utilized whenever temperatures of water were involved for the Punjab and Sind canals. This base temperature probably introduces some scatter in those relationships in which it is involved.

## RELATIONSHIP BETWEEN SEDIMENT LOAD, FROUDE NUMBER, AND CHANNEL STABILITY

To understand the importance of sediment load and Froude number on channel stability it is necessary to consider the behavior of alluvial channels in complete detail.

*Regimes of Flow and Forms of Bed Roughness.*—In an alluvial channel the forms of bed roughness are a function of the bed material, the sediment in

SIMONS AND BENDER DATA

| Bed Material, mm | | | Side Material, mm | | | Estimated Dune Height | Sediment Transport, in tons/day | |
|---|---|---|---|---|---|---|---|---|
| $d_{15}$ | $d_{50}$ | $d_{85}$ | $d_{15}$ | $d_{50}$ | $d_{85}$ | | Suspended | Total |
| 0.240 | 0.580 | 1.140 | 0.062 | 0.207 | 0.462 | 0.10 | 47.1 | 213.5 |
| 0.123 | 0.208 | 1.500 | 0.029 | 0.133 | 0.264 | 0.30 | 319.5 | -- |
| 0.124 | 0.253 | 0.527 | 0.039 | 0.087 | 0.147 | 0.50 | 103.2 | 320.0 |
| 0.022 | 0.096 | 0.274 | 0.017 | 0.0515 | 0.161 | 0 | 156.8 | 443.0 |
| 0.0054 | 0.0287 | 0.142 | 0.011 | 0.0419 | 0.136 | 0 | 151.0 | -- |
| 0.388 | 0.805 | 1.230 | gravel | gravel | gravel | 0.5 | -- | -- |
| 0.146 | 0.318 | 0.545 | 0.038 | 0.0806 | 0.191 | 0.1 | 20.05 | 100.0 |
| 0.316 | 0.617 | 1.280 | 0.024 | 0.098 | 0.506 | 0.5 | 22.90 | -- |
| 0.232 | 0.390 | 0.685 | 0.017 | 0.098 | 0.295 | 0.4 | 38.70 | -- |
| 0.161 | 0.465 | 0.945 | 0.030 | 0.143 | 0.479 | 0.4 | 40.30 | 28.8 |
| 0.294 | 0.568 | 1.240 | 0.029 | 0.166 | 0.499 | 0.5 | 51.30 | -- |
| 0.290[a] | 7.00[a] | 25.00[a] | 0.031 | 0.109 | 0.327 | 0 | 72.10 | 235.0 |
| 0.370[a] | 7.60[a] | 20.00[a] | 0.027 | 0.060 | 0.141 | 0 | 102.5 | -- |
| 0.178 | 0.311 | 0.848 | 0.071 | 0.149 | 0.247 | 0.4 | 157.2 | 516.0 |
| 0.311 | 0.575 | 1.010 | 0.014 | 0.074 | 0.180 | 0.4 | 90.3 | -- |
| 0.051 | 0.173 | 0.619 | 0.031 | 0.079 | 0.121 | 0.1 | 7.8 | 36.8 |
| 0.052 | 0.163 | 0.422 | 0.040 | 0.077 | 0.239 | 0.1 | 5.6 | -- |
| 0.053 | 0.229 | 0.648 | 0.037 | 0.182 | 0.266 | 0.1 | 1.7 | 47.0 |
| 0.327 | 0.715 | 1.648 | 0.074 | 0.286 | 0.573 | 0.1 | 44.3 | 65.6 |
| 0.013 | 0.360 | 0.117 | 0.014 | 0.036 | 0.056 | 0 | 88.8 | 130.5 |
| 0.015 | 0.349 | 0.158 | 0.014 | 0.034 | 0.069 | 0 | 16.8 | 13.38 |
| 0.220 | 0.446 | 1.327 | 0.043 | 0.177 | 0.782 | 0.20 | 55.8 | 61.0 |
| 0.219 | 0.420 | 0.993 | 0.073 | 0.271 | 0.995 | 0.60 | 97.3 | -- |
| 0.147 | 0.246 | 0.495 | 0.026 | 0.067 | 0.188 | 0.30 | 45.9 | -- |

transport, and the flow. Based on the foregoing field study and a laboratory study of alluvial channels the following regimes of flow and forms of bed roughness have been observed.

Tranquil flow regime ($F_r < 1$)

1. Plane bed without movement of bed material;
2. Ripples;
3. Ripples superposed on dunes;
4. Dunes;
5. Transition from dunes to rapid flow;
6. Plane bed with movement of bed material; and

Rapid flow regime ($F_r < 1$);

7. Standing water waves and sand waves; and
8. Antidunes.

These forms of bed roughness have been described by M. L. Albertson, D. B. Simons, and E. V. Richardson (1).

*Sediment Load.*—The magnitude of the total silt-sand sediment load varies with form of bed roughness, In accordance with laboratory data (1), and in ac-

cordance with data from the field studies, the relationships between total sand-silt transport and form of bed roughness is, for sand beds of well graded material having a median diameter of $d \geq 0.2$ mm, roughly as follows:

| Flow Regime and Form of Bed Roughness | Sediment Load, ppm |
|---|---|
| Tranquil flow regime | |
| Ripples | 0 to 90 |
| Dunes | 90 to 1,000 |
| Transition of dunes to rapid flow | 1,000 to 3,000 |
| Plane bed with movement of bed material | 2,000 to 5,000 |
| Rapid flow regime | |
| Standing waves, Antidunes | In excess of 4,000 |

It should be emphasized that these limits of total load are qualitative and will be influenced by many factors—such as the characteristics of the bed material, channel geometry, shear on the bed, and characteristics of the suspended sediment. For example, considering Kalinske and Hsia data (1), the total load was on the order of 17,000 ppm with dunes. However, in this case, the median diameter d = 0.011 mm. Very fine suspended sediment which is sometimes referred to as wash load has been defined as that material transported by the flow which is not found in appreciable quantities in the bed. As a result of field observations (Figs. 3, 5, and 8, to be presented subsequently) it is apparent that channel stability is very intimately related to wash load when wash load concentrations are large. The foregoing figures illustrate that with the ripple and dune forms of bed roughness the presence of very fine sediment reduces resistance to flow and, also, drastically influences channel geometry. The effect of fine sediment on fluvial mechanics is sufficiently important to warrant further study.

*Channel Stability as Related to Froude Number.*– The field study of stable channels conducted by the writers and subsequent discussions of channel stability with others has emphasized that if a channel is to be stable (no appreciable bed and/or bank scour or accretion occurring with time) the Froude number defined as $V/\sqrt{g\,D}$, must in most cases be less than 0.3 for alluvial material in the sand-size range and finer. When $F_r > 0.3$, the banks scour in the straight reaches of channel and the bends must be stabilized to confine them to their rights-of-way.

*Summary.*—Summing up the preceding concepts as they apply to design of stable channels, it can be concluded, for alluvial materials in the sand range and finer, that if excessive bank erosion is to be avoided the Froude number must not exceed 0.30. This immediately confines the design problem to the tranquil flow regime and excludes the transition and plane bed form of bed roughness within this regime because of the magnitude of the Froude number associated with it. That is, the designer is restricted such that the channel should be designed with the ripple, or dune form of bed roughness, depending

on channel slope limitations as dictated by the terrain, and the magnitude of sand load which the channel must transport. Once it is recognized that designs must be confined to a range of small Froude numbers and to the forms of bed roughness cited, it is immediately apparent that the magnitude of sand-silt sediment load which can be transported is, likewise, severely limited—usually to less than 500 ppm.

This discussion brings out still another important point. Canals must utilize head works structures which do not allow sediment concentrations, in the sand-silt range, to exceed 500 ppm for the dune form of bed roughness, and the concentration of sediment must be even smaller for ripples. Otherwise the channel will be unstable—aggrading with time.

If channels are designed to carry larger sediment loads (concentrations > 500 ppm) the channel will be unstable because of banks and bend erosion ($V/\sqrt{g\,D} > 0.3$) unless some form of bank stabilization is utilized. It cannot be overemphasized that if one must transport more sediment than can be handled within the dune regime it will be necessary to provide bank stabilization such as a gravel rip-rap, or some other form of armor plating, or masonry or concrete. The tractive force method of design, as presented by Lane (13), provides a satisfactory means of designing stable protection, using coarse noncohesive material for this condition.

## ANALYSIS OF DATA

The tabulated basic data and the parameters computed therefrom can now be utilized to investigate the theory of stable channels. The shape characteristics of these canals will be investigated first.

*Relationship Between R and D, and P and W.*—The Lacey theory is expressed in terms of wetted perimeter and hydraulic radius. The Blench theory is in terms of average depth on the channel bed and average width.

Considering the 42 Punjab canals the average depth on the bed and average width were not given. The only measurements pertaining to shape were wetted perimeter, hydraulics radius, and top width. To overcome this deficiency of data, and for design purposes, hydraulic radii were correlated with average bed depths and wetted perimeters were correlated with average width of channel, as shown in Figs. 1 and 2. Data from the 24 canals investigated by the writer, the 28 Sind canals, and the Imperial Valley canals were utilized to establish these curves.

The Imperial Valley canal data have been plotted on Figs. 1 and 2 to show that increased charge has little effect on the relationships between R and D, and W and P.

Based on Figs. 1 and 2, it is now obvious that average width and bed depth can be estimated rather accurately if P and R are known.

The actual method used to compute W and D for the 42 Punjab canals involved estimating W, knowing P, and then computing average bed depth by means of equation A = W D.

*Estimating P and R.*—According to the Lacey and Blench theories, it is to be expected that either wetted perimeter or some channel width dimension, such as top width or average width, should correlate with rate of discharge. Based on this type of correlation, Lacey arrived at Eq. 9.

*Estimating P.*—Using Lacey's procedure, values of P and corresponding values of Q were plotted in Fig. 3, first for the canals investigated by Simons

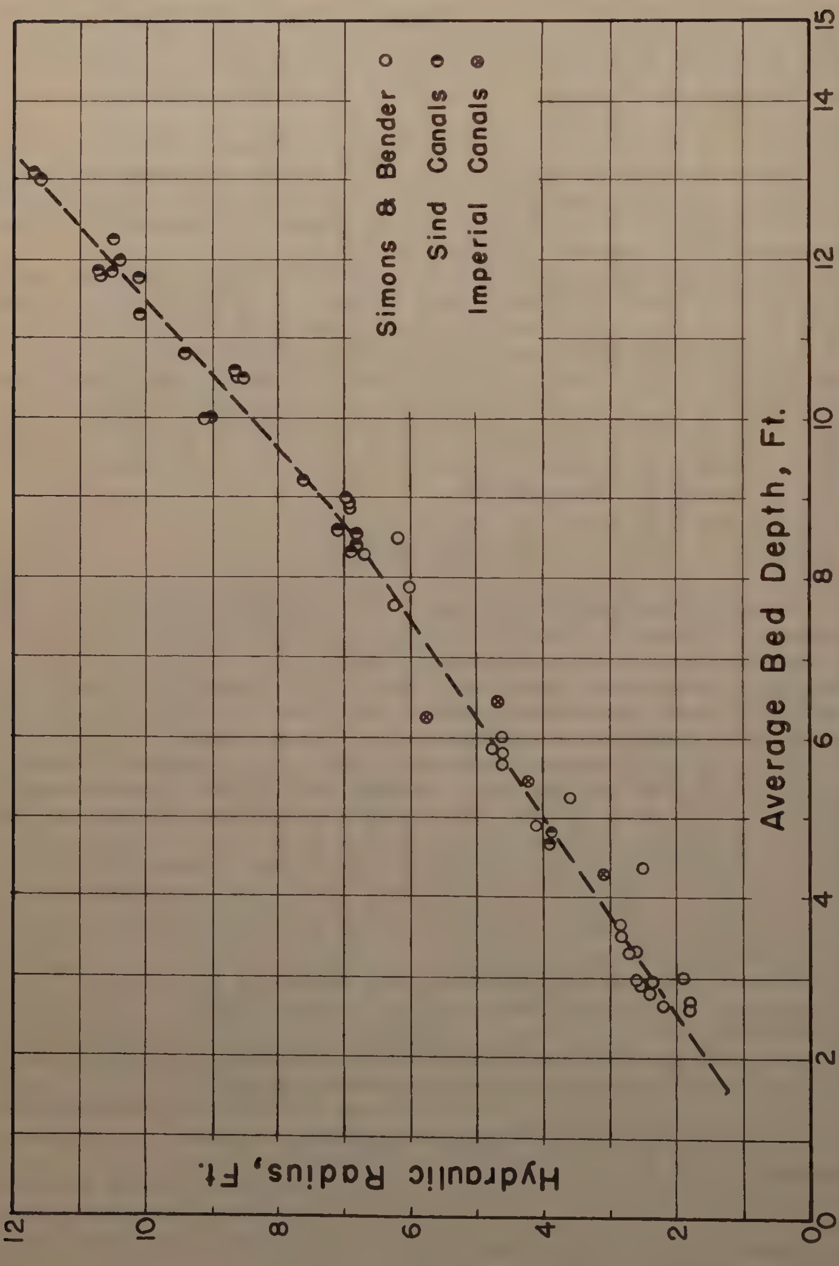


FIG. 1.—VARIATION OF HYDRAULIC RADIUS R WITH DEPTH D

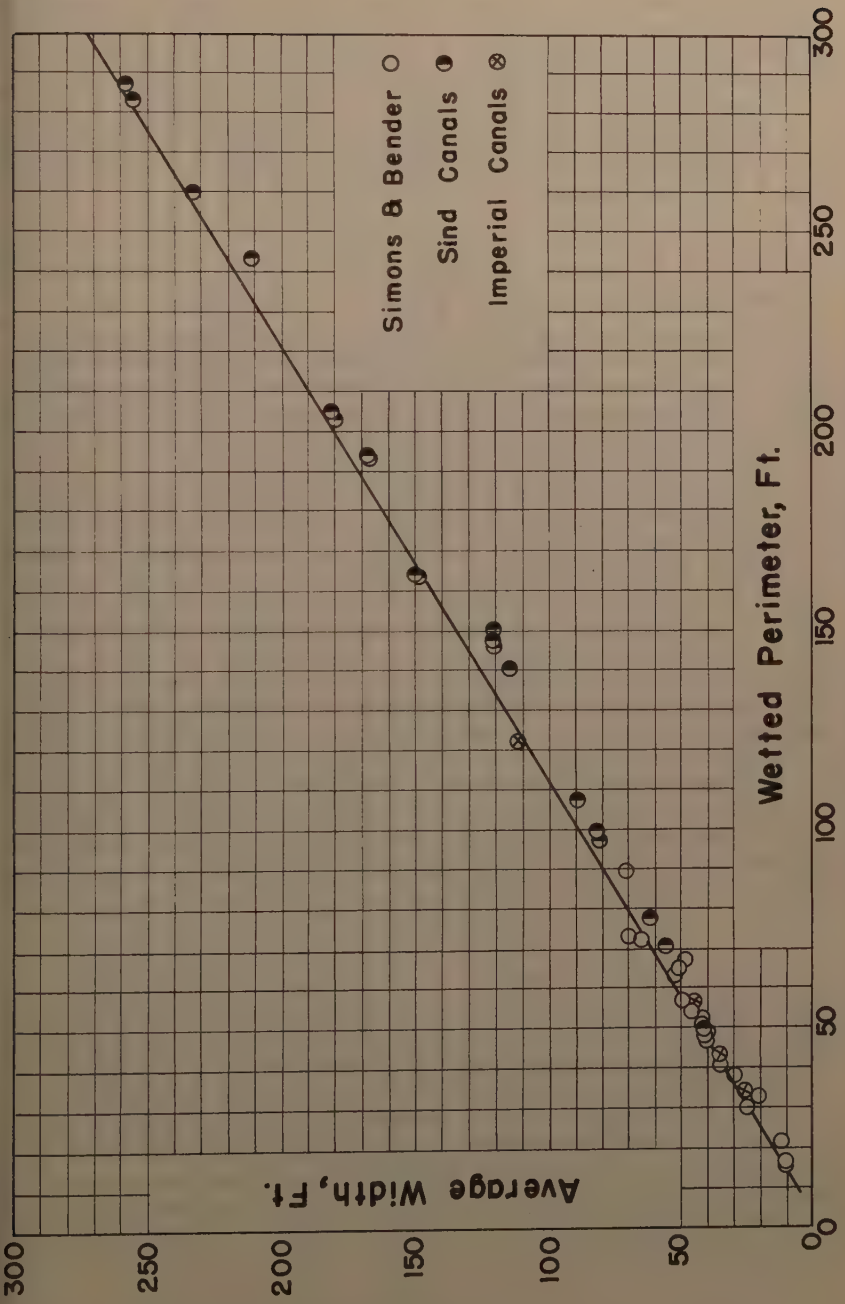


FIG. 2.—VARIATION OF AVERAGE WIDTH W WITH WETTED PERIMETER P

and Bender (referred to as Simons and Bender data). The range of materials forming the periphery of these canals extends from fine cohesive material to coarse non-cohesive material. The effect of soil type on P is clearly exhibited in Fig. 3. The sand channels all require a relatively large P for a given Q, while cohesive materials reach stability at a relatively small P for a given Q.

To illustrate more fully how the 24 Simons and Bender data plot relative to the India data, values of P versus Q have also been plotted in Fig. 3 for the forty-two Punjab canals. Three curves have been fitted to these data based on type of bed and bank material. The equation of the arbitrary straight line representing canals with sand beds and cohesive banks is

$$P = 2.5\ Q^{0.51} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (19)$$

Note the similarity of Eqs. 9 and 19.

The effect of bank and bed materials on such a relationship is illustrated further by considering the USBR data. With these data it is seen that values of P for a given Q are even smaller than in the case of cohesive materials. This simply illustrates the ability of the coarse material to resist a greater tractive force.

The Imperial Valley canal data have also been plotted to illustrate the quantitative effect of increased charge. From the viewpoint of type of bank and bed materials, these canals are similar to the other canals, excluding those formed in coarse non-cohesive material. The trend line, however, falls approximately on the trend line representing the relationship between P and Q for canals formed in coarse non-cohesive material. This indicates that as a fine sediment load, of the type found in the Imperial canals decreases, the stable wetted perimeter, P, and consequently stable width, W, decreases.

*Relationship Between Average Width and Top Width.*—In most of the relationships involving width, W, the average value has been used. Under certain circumstances, it may be advantageous to convert average width, W, to top width, $W_T$. To facilitate this conversion, see Fig. 4. This relates W with $W_T$, based on the data from the canals investigated by Simons and Bender and the Punjab canal data. A good straight-line correlation exists between these variables up to a top width of approximately 300.0 ft. However, for the larger values of W, the relationship is based on limited data. The equation relating these variables is

$$W = 0.92\ W_T - 2.0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (20)$$

*Estimating R.*—A relationship between D and R for stable irrigation canals was given in Fig. 1. It is now desirable to relate R to other quantities or parameters. Lindley showed that both D and R were very closely related to Q. The truth of this for R is illustrated in Fig. 5. In this case, values of R and Q for all canals have been utilized, as in the foregoing relation, between Q and P.

The effect of natural bed and bank material is apparent and consistent with the preceding relations. The canals constructed in cohesive materials fall in one group, those having a sand bed and natural berm in a second group, those in sand material in a third group, and those in coarse non-cohesive material fall in a fourth group.

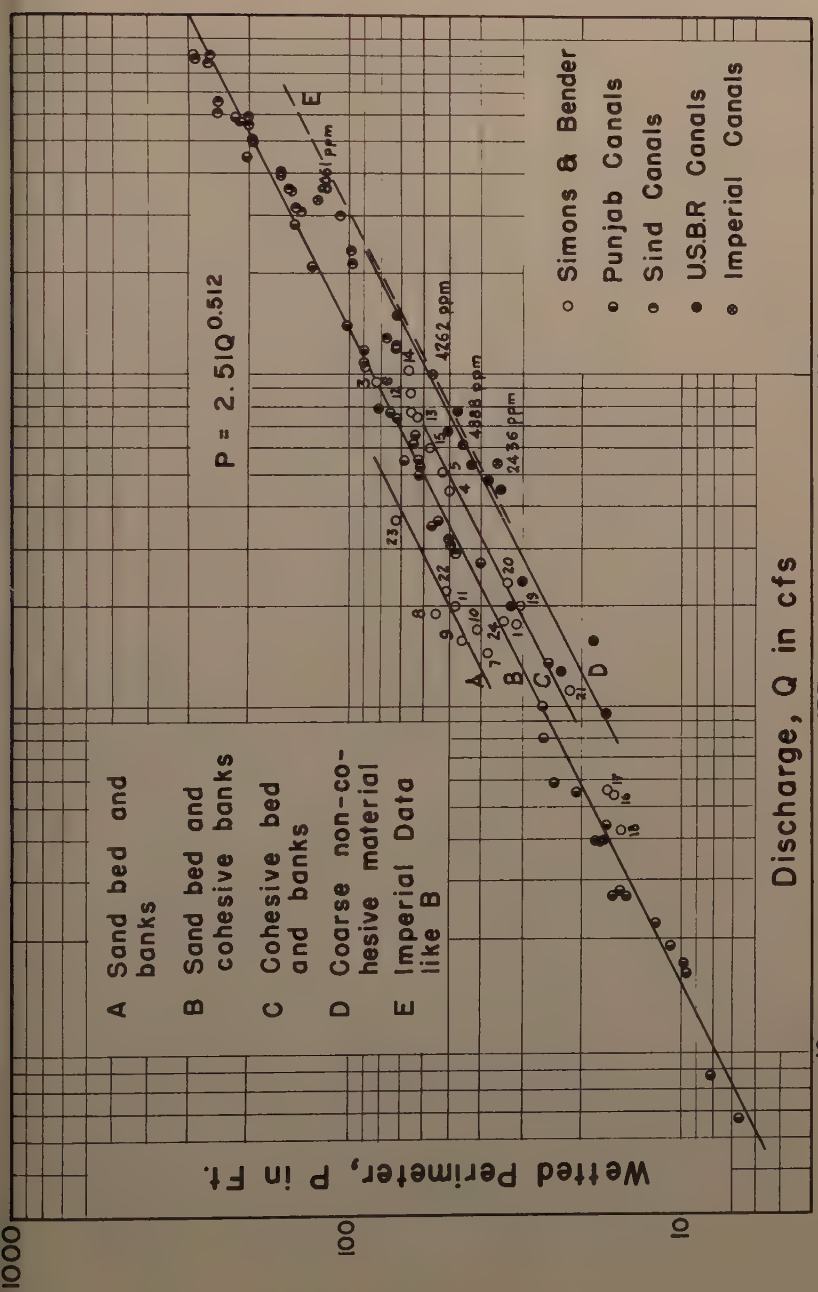


FIG. 3.—VARIATION OF WETTED PERIMETER P WITH DISCHARGE Q AND TYPE OF CHANNEL

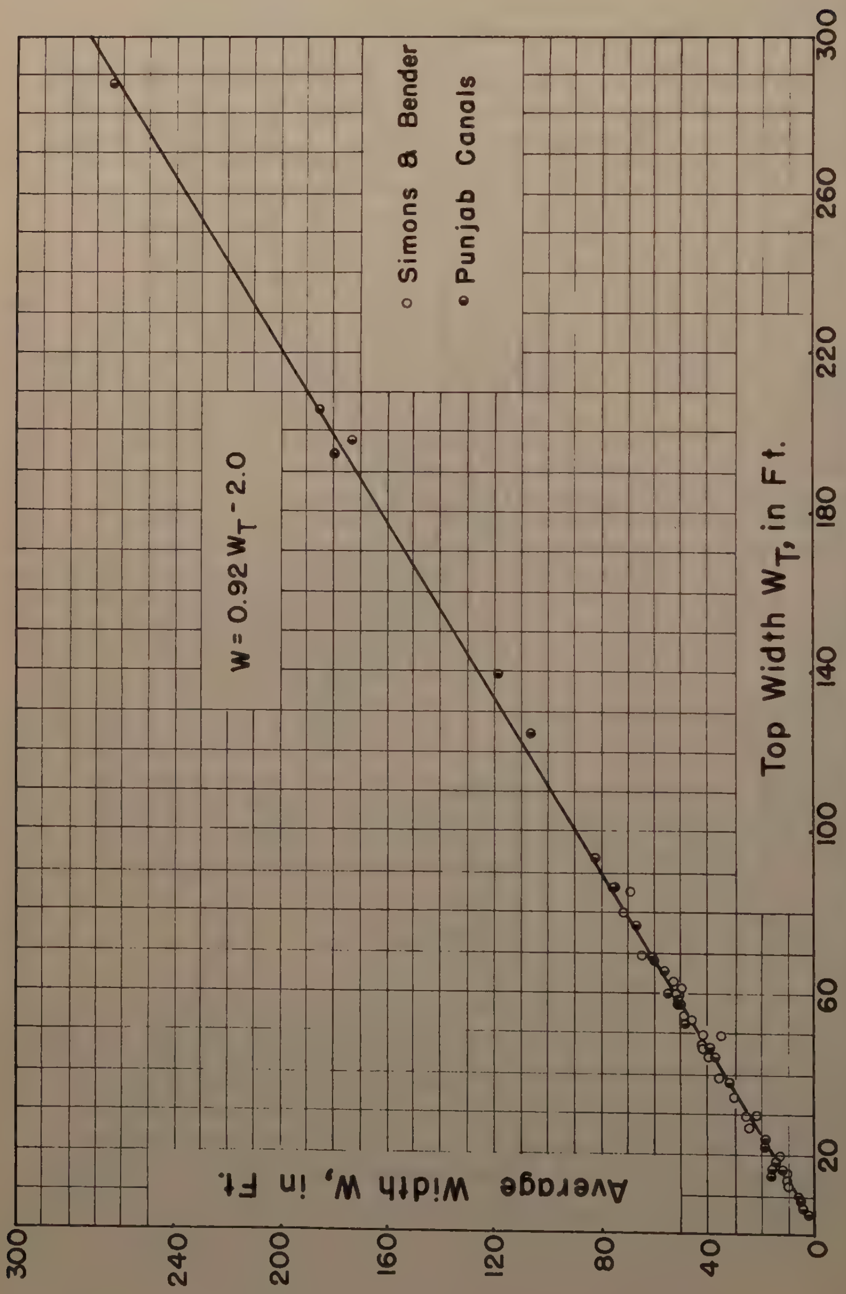


FIG. 4.—VARIATION OF AVERAGE WIDTH W WITH TOP WIDTH $W_T$

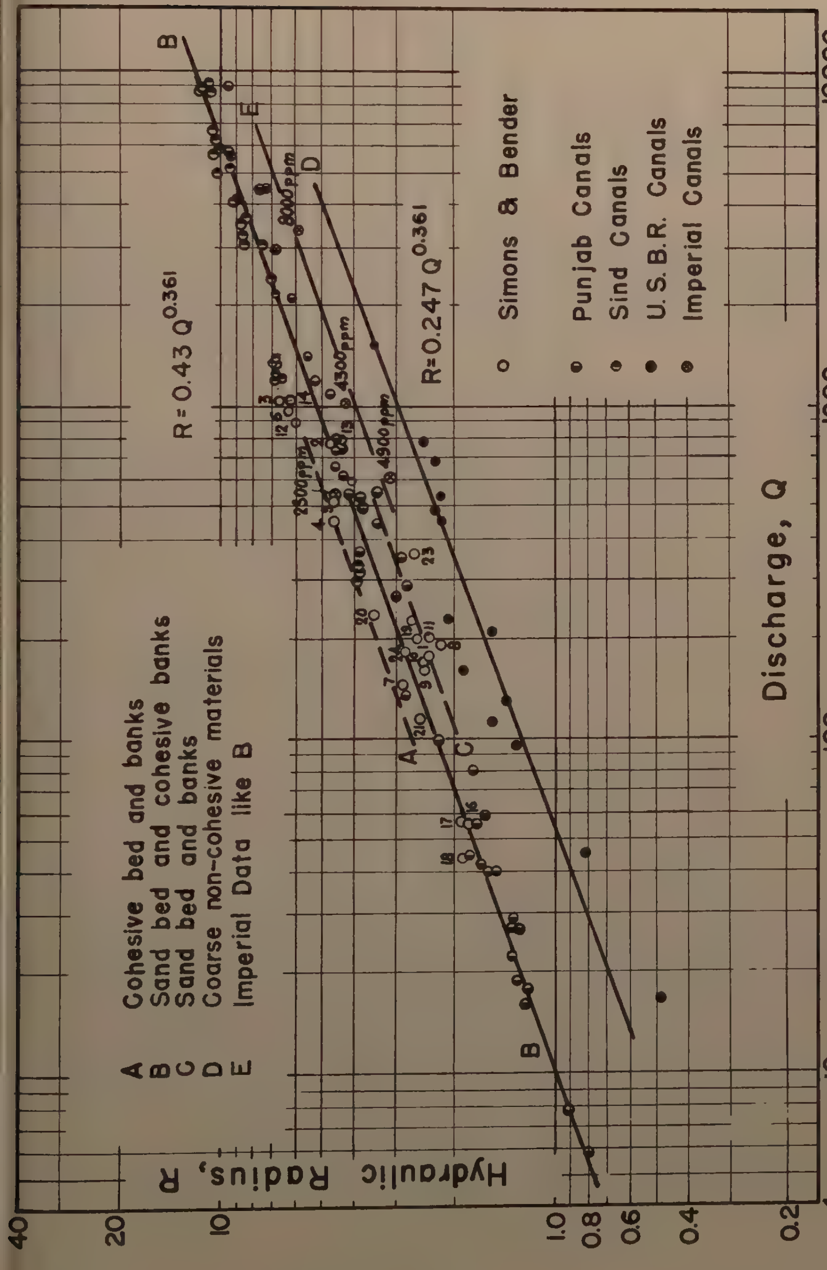


FIG. 5.—VARIATION OF HYDRAULIC RADIUS R WITH DISCHARGE Q AND TYPE OF CHANNEL, ALL DATA

The basic relationship relating R and Q for canals having a sand bed and natural berm is

$$R = 0.43\ Q^{0.361} \qquad (21)$$

The same type of relationship for the canals in coarse non-cohesive material is

$$R = 0.247\ Q^{0.361} \qquad (22)$$

The influence of sand banks on R has not been expressed in equation form but their effects are clearly illustrated.

The Imperial Valley canal data have been plotted to illustrate the importance of charge on stability of channels (Fig. 5). Comparing the trend line for these limited data with similar lines representing the Simons and Bender, Punjab, and Sind data, it is apparent that the hydraulic radius, R, decreases with increased charge for a given discharge, Q.

*Relationship Between Q and A.*—It is of interest to note the close relationship existing between Q and A for stable channels. Once again, however, it is expedient to introduce type of bank material as a third variable (Fig. 6). Values of Q and A have been plotted from all canals. Four curves have been drawn, as indicated by bank material and bed condition. The short, uppermost curve is for canals having sand banks. The intermediate curve is for all other bank materials finer than sand. The lower curve is for coarse non-cohesive materials, as represented by the USBR canal data.

Equations relating A and Q for two of the foregoing three conditions follow. For banks of material finer than sand extending into the very cohesive range

$$A = 1.076\ Q^{0.873} \qquad (23)$$

and for coarse non-cohesive materials

$$A = 0.45\ Q^{0.873} \qquad (24)$$

The relationship between Q and A for the Imperial Valley canal data shows that as magnitude of charge increases the required area, A, corresponding to a given discharge, Q, decreases and the permissible average velocity, V, is increased. This is consistent with the trends indicated in Figs. 3 and 5. It is also important to note in this case, the variation of vertical displacement of each point relative to the line representing sand beds and cohesive banks. The effect of variation of charge shows up very clearly. These expressions cover a wide range of discharge and boundary conditions and should be very useful in practical design work.

*Expressions Involving Velocity, Hydraulic Radius and Slope.*—A variety of regime-type equations have been, recommended to determine design slope by one individual or another. One of the most significant is the Lacey formula

$$V = 16.0\ R^{2/3}\ S^{1/3} \qquad (25)$$

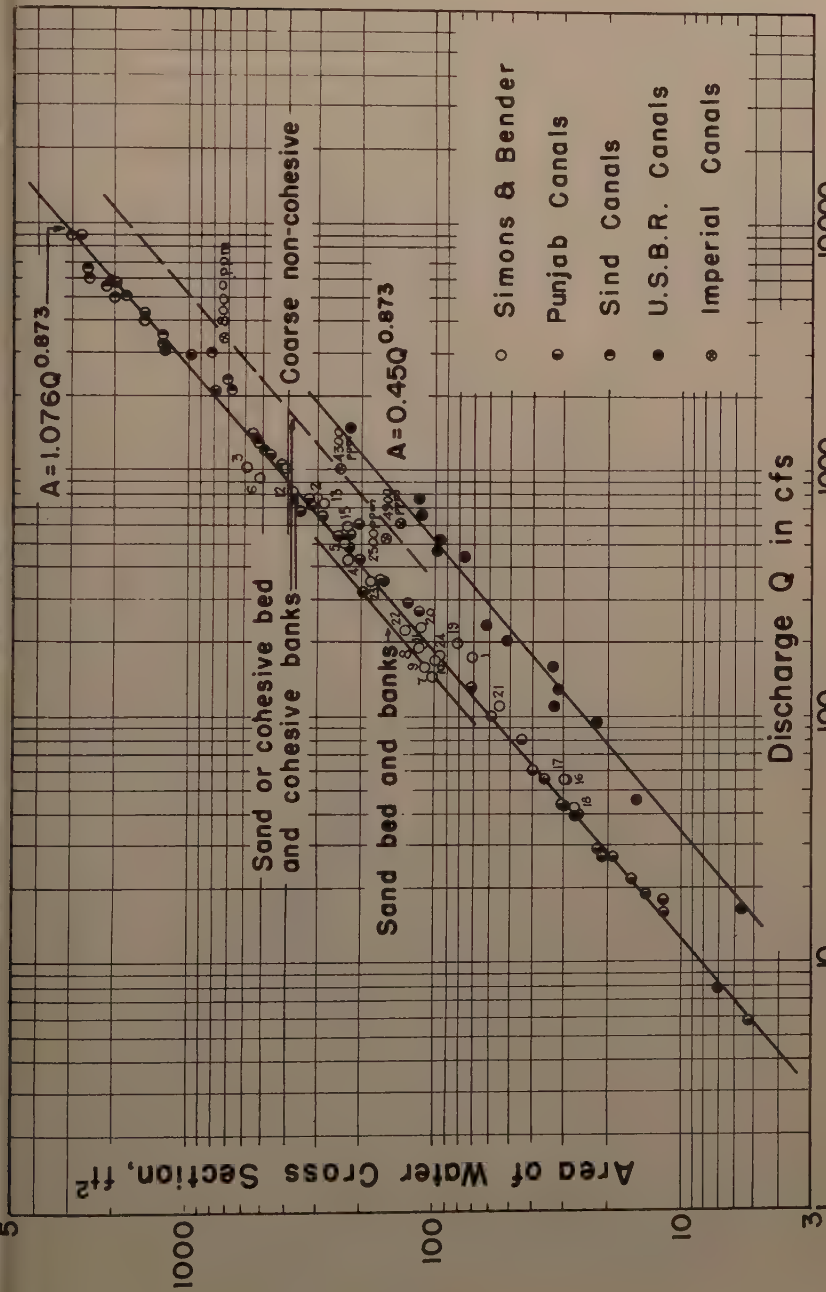


FIG. 6.—VARIATION OF AREA OF WATER CROSS-SECTION A WITH DISCHARGE Q AND TYPE OF CHANNEL

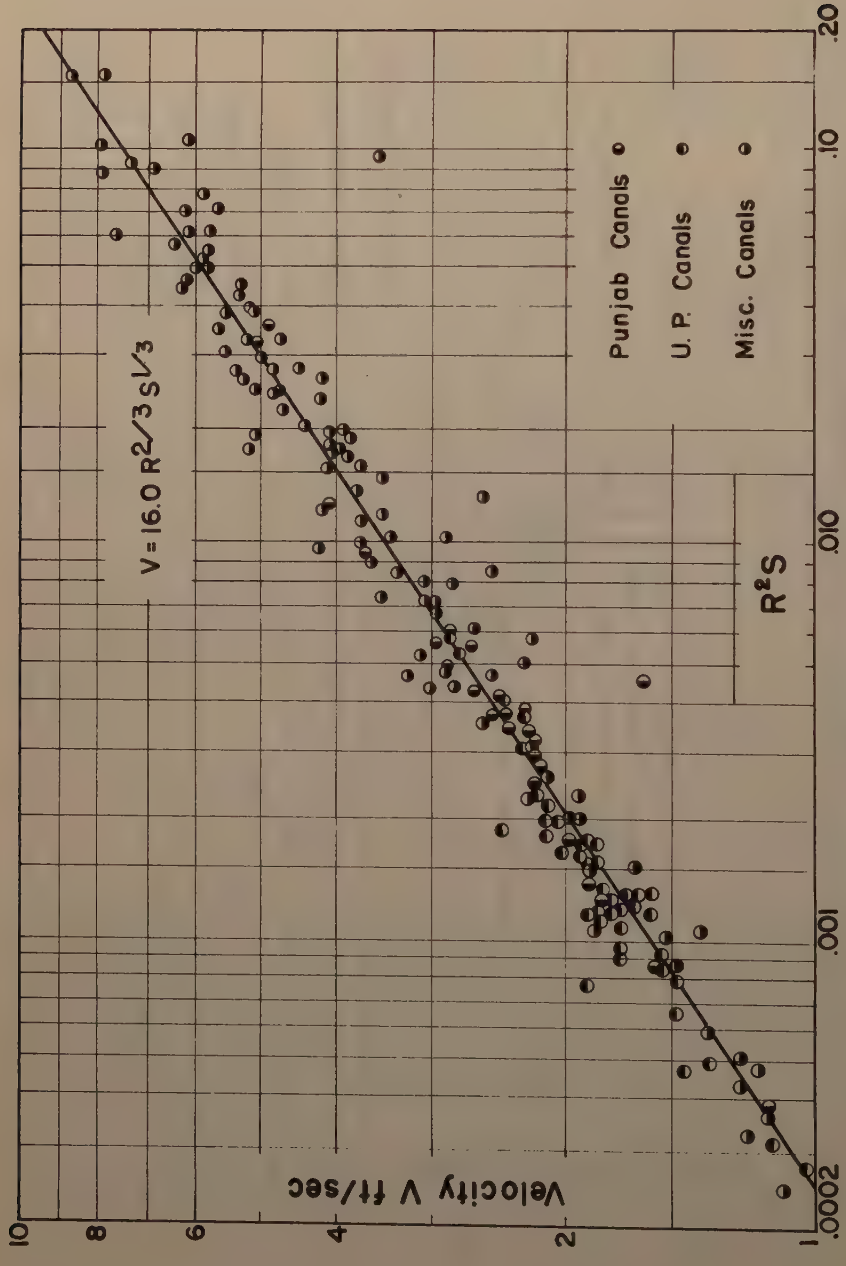


FIG. 7.—VARIATION OF AVERAGE VELOCITY V WITH $R^2S$, INDIA DATA

The general validity of this expression is shown in Fig. 7 (9). An extremely wide range of discharge is covered. Data used include the Punjab data and other miscellaneous India canal data which are not presented or utilized elsewhere in this report. The difficulty with this expression is that, although the trend is very definite, some slopes computed by this relationship vary considerably from the measured slopes used to establish the correlation.

To serve as a further check on Fig. 7, V versus $R^2$ S has been plotted in Fig. 8 for all the canals included in this report. These data fall in three separate groups in the plot, and a line has been drawn through each group. The upper line correlates V with $R^2$ S for the coarse non-cohesive materials represented by the USBR canal data. The intermediate line does the same for canals with sand beds and slightly cohesive to cohesive banks. The third line represents sand bed conditions, at least insofar as the Simons and Bender data are concerned. The equations of these lines are:

$$V = 17.9 \left(R^2\ S\right)^{0.286} \qquad (26)$$

$$V = 16.0\ R^{2/3}\ S^{1/3} \qquad (27)$$

and

$$V = 13.86 \left(R^2\ S\right)^{1/3} \qquad (28)$$

respectively. Eq. 27 is identical with the preceding Eq. 25 corresponding to the plot of Fig. 7.

The Imperial Valley canal data show that for a given value of $R^2$ S, the permissible average velocity, V, increases as charge increases and knowing R and V, from Figs. 5 and 6, the slope consistent with stability can be estimated.

*The Blench-King Regime Slope Formula.*–The regime slope equation recommended by Blench for design is

$$S = \frac{b^{5/6}\ s^{1/12}}{2080\ r\ Q^{1/6}} \qquad (29)$$

This was derived by plotting W/D against a variety of non-dimensional groups and would have been found at once, according to Blench, by plotting $V^2/g$ D S against V $W/\nu$. The basic regime slope formula is

$$\frac{C^2}{g} = \frac{V^2}{g\ D\ S} = \left(\frac{V\ W}{\nu}\right)^{1/4} \qquad (30)$$

where W is the average channel width.

In Fig. 9, values of $V^2/g$ D S have been plotted against V $W/\nu$. A value of $\nu$, corresponding to 70 F, has been assumed for all India canals. The Punjab canals yield points that plot quite close to a straight line on log-log paper between the limits of $10^5 < V\ W/\nu < 10^7$. Beyond this upper limit, the $V^2/g$ D S terms are nearly constant and the slope of the line flattens until it lies approximately parallel to the horizontal axis.

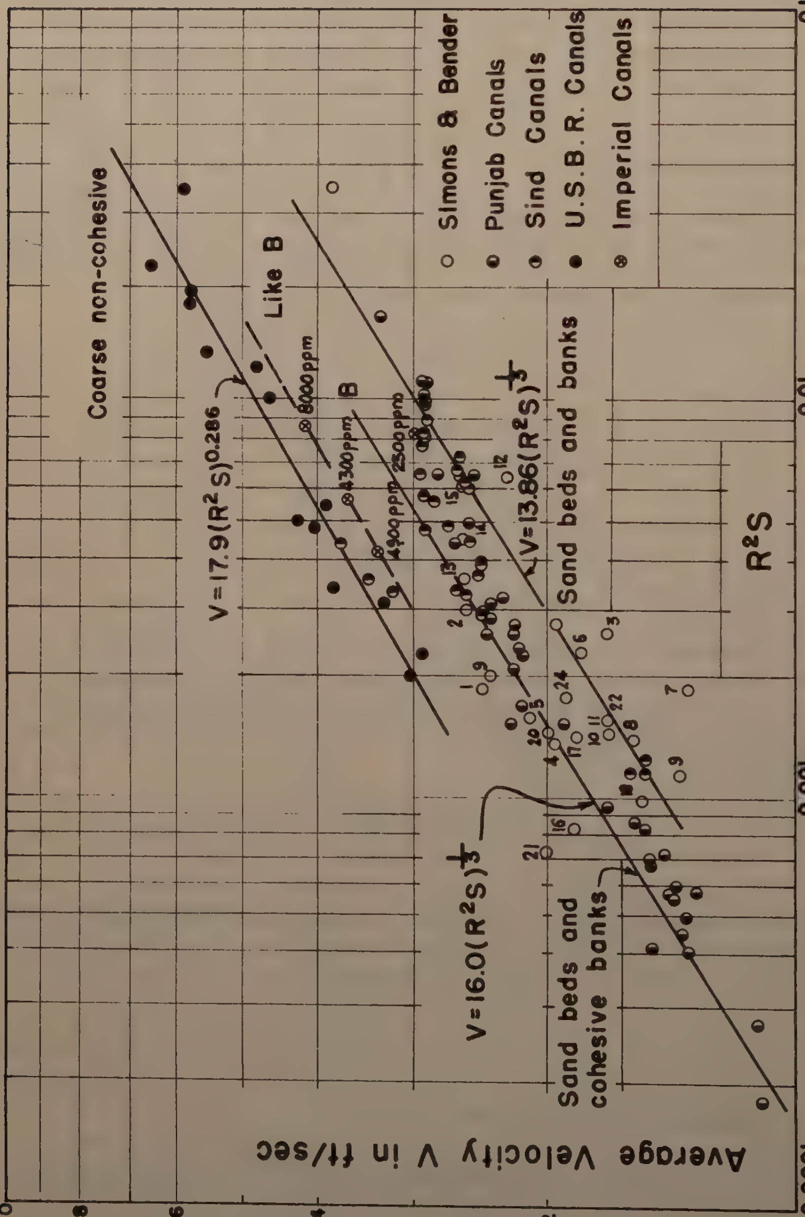


FIG. 8.—VARIATION OF AVERAGE VELOCITY V WITH $R^2S$ AND TYPE OF CHANNEL, ALL DATA

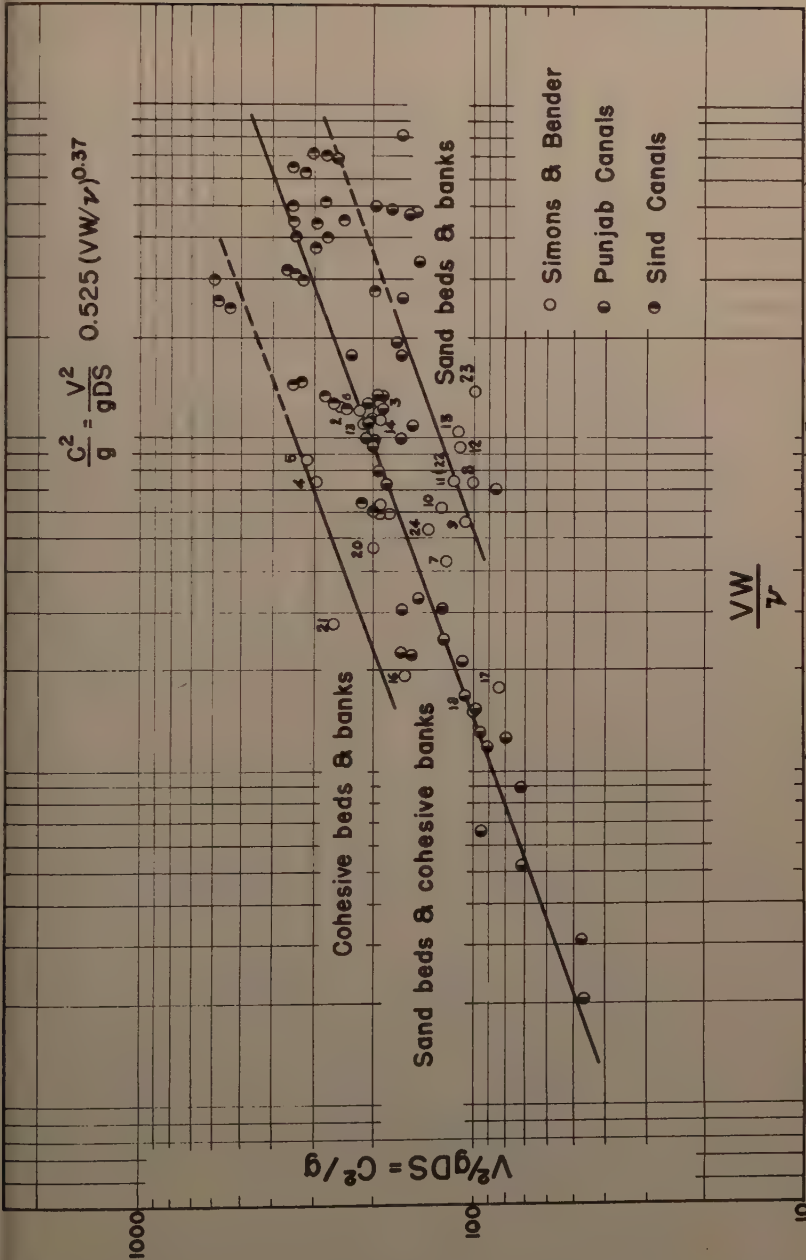


FIG. 9.—VARIATION OF $V^2/gDS$ WITH $VW/\nu$ AND TYPE OF CHANNEL

The twenty-eight points corresponding to the Sind data lie more or less on an extension of the straight-line portion of the Punjab data.

The Simons and Bender data give points that generally intermingle with the India canal data, except that seven of the points corresponding to canals with non-cohesive banks fall lower, as one would anticipate.

The significance of the basic regime slope equation, presented by King and Blench, is quite apparent from study of Fig. 9. The India data as a group plot close to a straight line between the limits of $10^5 < V\ W/\nu < (5)^8$. Beyond $W\ V/\nu = (5)^8$, more points fall below the straight line than above it, however, this may be a function of the canals sampled.

*Tractive-Force Relationships.*—The various procedures used to estimate magnitude of tractive force on the bed and sides of channels were discussed. Values of shear on the channel periphery were computed for the 24 canals investigated by Simons and Bender, using all of the methods described. A summary of these data is given in Table 3.

*Magnitude of Tractive Force.*—The magnitude of boundary shear varies with method of computation. In Fig. 10, a typical canal cross section is given, including the shear distribution on its boundary, as indicated by the various methods of computation, size of side and bed material, and standard deviation of side and bed material. As illustrated in Fig. 10, shears computed by the various methods for a given channel are by no means in close agreement. In general, shears computed by Eq. 16, and shears computed by use of isovels and the concept of zero momentum transfer are both larger than shears indicated by the velocity gradients measured normal to the boundary across the channel. The lack of agreement in results, and the fact that shears based on velocity gradients are smaller, leads one to believe that something is being neglected when shears are computed based on the latter method. It may be that this results because the energy required to transport the sediment load and/or the energy involved in secondary circulation are being neglected.

Another interesting observation based on shears computed from knowledge of velocity distributions is the way shears vary across the bed section of each canal. Generally, one might expect a more uniform shear distribution such as is indicated by computations based on $\tau = \gamma\ D\ S$. This variation in distribution may be intimately related to secondary circulation in the canals. It has been proposed that secondary circulation be studied in the canals investigated by Simons and Bender at some future date to provide a better knowledge of its effect on shear and shear distribution, sediment transport, and channel stability in general.

*Correlation of Tractive Force With Mean Diameter of Bed Material.*— Lane (14), has developed a relationship which relates size of bed material and tractive force. This correlation provides a very useful means of establishing the design slope of channels and canals in coarse non-cohesive materials, provided size of bed and bank material can be estimated with reasonable accuracy.

By working in terms of tractive force based on D and/or R, the effect of the Punjab and Sind canal data on the foregoing relations can be shown. In Fig. 11, values of $\tau = \gamma\ R\ S$ and corresponding values of mean diameter of bed material have been plotted for all of the canals involved in this study, that is, the Simons and Bender data, the USBR data and the India data.

Several facts of interest are immediately apparent in Fig. 11. First a general line extending through all of the data can be drawn. There is, however, considerable scatter about this line. Next, secondary lines crossing the major trend line have been drawn based on an intimate knowledge of the Simons and

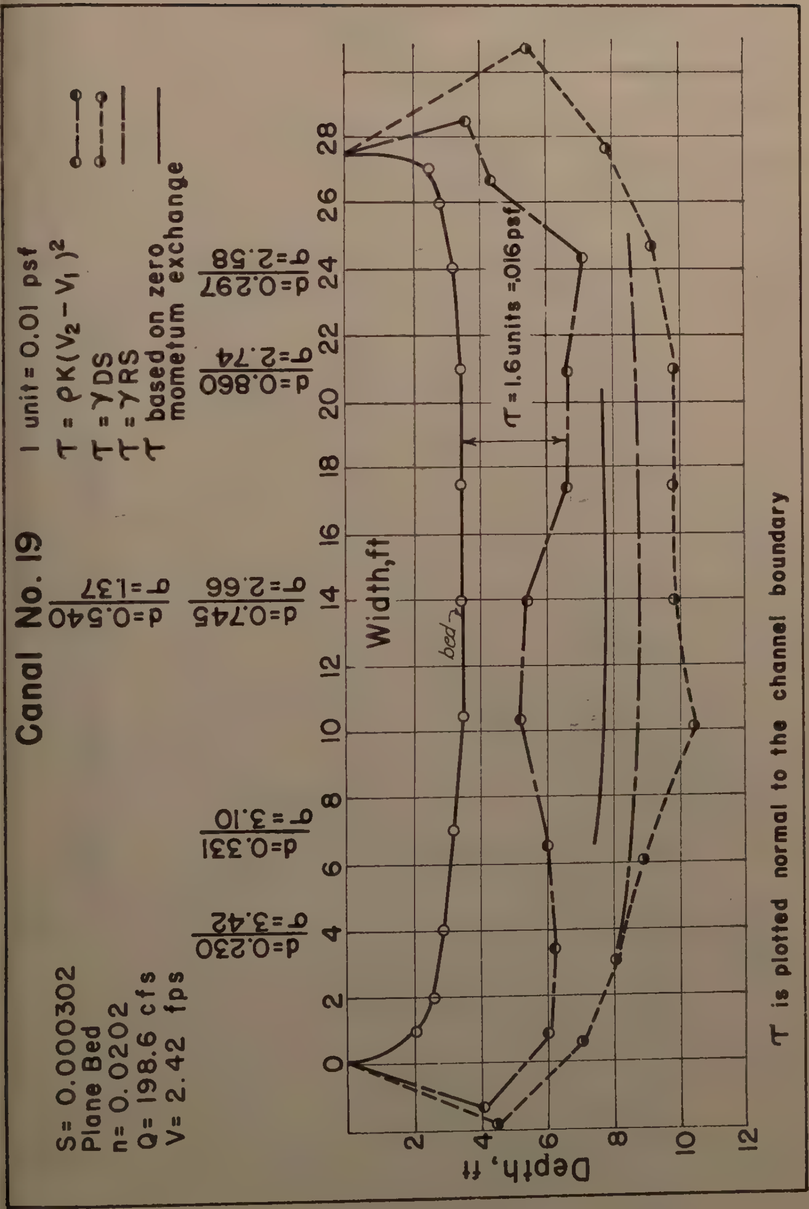


FIG. 10.—TRACTIVE FORCE DISTRIBUTION, CANAL NO. 19

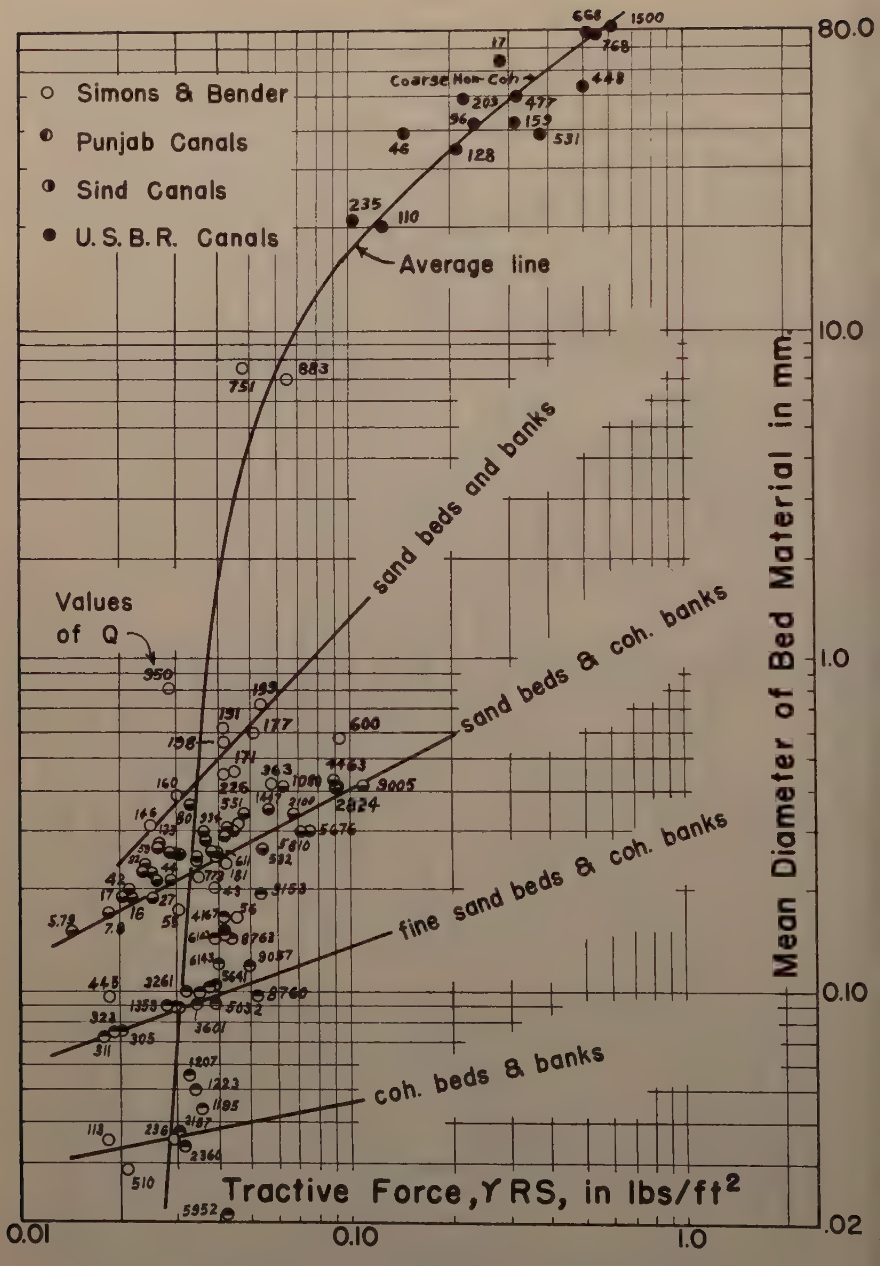


FIG. 11.—VARIATION OF TRACTIVE FORCE WITH BED MATERIAL d TYPE OF CHANNEL AND DISCHARGE Q

Bender data and a limited knowledge of the India data from a study of the literature and existing data. Moving in the upward direction, the first of these lines is associated with canals having cohesive beds and banks, the second with canals having fine sand beds and probably berm banks or natural banks of a cohesive nature, the third with canals having coarser sand beds and berm banks or banks of slightly cohesive natural materials, the fourth with canals having sand beds and banks, and the fifth with coarse non-cohesive beds and banks. Roughness of bed seems to increase traveling from the bottom secondary line to the fourth secondary line associated with sand beds and banks.

Next consider each of the five secondary lines. Moving along these lines in the direction of increasing shear, it is found that canal capacity increases. The points on the extreme right end of the secondary lines correspond to large Q-values, and the points at the extreme left of these same lines correspond to small Q-values. The whole system of lines shown in Fig. 11 are placed rather arbitrarily and would undoubtedly shift slightly if additional new data were incorporated into the plot.

The Imperial Valley canal data have not been plotted in Fig. 11 because of uncertainty regarding the mean size of bed material. An effort is currently being made to secure these data, since it will be of importance to reflect the effect of increasing the magnitude of charge on permissible tractive force.

Considering the lowest element of the major curve, it is noted that if it were curved to the right it would fit the plotted points somewhat better. This indicates that allowable tractive force probably increases with a decrease in mean size of sediment smaller than that size where the material starts to become plastic. This aspect of the problem was investigated by a flume study at the University of Wyoming (23). The flume study verified that limiting tractive force increases with increase in plastic index. The investigation was carried out under the supervision of the writer using natural materials from canal beds investigated by Simons and Bender.

*Channel Shape.*—Considering the canals investigated by Simons and Bender it was observed that channel shapes varied widely. It was also apparent that shape was appreciably affected by the type of natural bank material, the amount and type of bank vegetation, and type and concentration of wash load. Most of these canals, excluding those found in sand, had sides that were very steep near the water surface (this was possible because of the reinforcement provided by the plant roots) and that were asymptotic to the channel bed.

The shape of these 24 canals, with some exceptions, conform reasonably well to the theoretical regime channel shapes as described by King Yu (24) and R. E. Glover (8).

Lane (13), and others have pointed out the possibility of designing channels to such shape and dimensions that the entire wetted perimeter is in a state of incipient motion. None of the canals investigated have adjusted to the foregoing shape, implying that it is perhaps necessary to construct to this form initially if it is desired. It seems that this aspect of design is worthy of a more thorough investigation.

*Angle of Repose for Non-cohesive Materials.*—In conjunction with selecting suitable side slopes when designing channels or ripraping canal banks, it is necessary to be able to estimate the angle of repose of the bank and/or rip-rap material. Lane (14), presented a figure relating angle of repose, median diameter of the material and shape of particles. Subsequent investigation of this relationship by Lane and Simons (25) verified that additional information on

angle of repose was needed. As a result of further study, Fig. 12 was developed. The angle of repose, as indicated in this figure, is for dry or saturated material. If the material is saturated and subjected to the action of flowing water as well, such as canal banks are, the side slope should be reduced below that indicated for static conditions by 5° to 10°.

## DESIGN PROCEDURES

The relationships of the previous paragraphs, regardless of their limitations, provides a useful design procedure. The objective of this section is to discuss the procedure and to point out the respective limitations and advantages.

In most cases, because of the general scatter of data, the complexity of the relations, and the time involved, curves have been fitted to the data visually. Where the scatter of the points about the trend line is not excessive, equations describing the relations have been determined.

*Selection of P.*—In Fig. 3, P, Q, and type and condition of bed and bank material have been related. The four parallel curves presented are representative of:

1. Sand beds and banks;
2. Sand beds and slightly cohesive to cohesive banks;
3. Cohesive beds and banks; and
4. Coarse non-cohesive beds and banks.

Although data for canals having sand banks are limited, Curve 1 provides a means of estimating P for this condition except possibly in the very small, and, also, in the very large, ranges of Q. Curve 2 is valid for the complete range of Q covered by the basic data and has been described mathematically by Eq. 19.

Curve 3 is based on very limited data and should be employed with this fact in mind.

Curve 4 is recommended as a means of estimating the magnitude of P in the coarse non-cohesive range of bed and bank materials.

At this point, knowing the wetted perimeter, the average stable channel width can be determined from Fig. 2.

*Estimating W Knowing P or Vice Versa.*—Wetter perimeter and top width are closely related, as illustrated in Fig. 2. Based on this relation, either P or W can be estimated for stable canals provided one or the other is known. The wetted perimeter, P, would undoubtedly correlate equally well with top width. The wetter perimeter estimated in this manner should be more representative of true conditions than if it were computed based on some initial trapezoidal shape.

*Selection of R.*—Referring to the relationships between discharge and hydraulic radius, Fig. 5, values of R corresponding to Q can be obtained directly from the appropriate curve or can be computed by equation if in the sand bed-cohesive bank or coarse non-cohesive range of materials. These equations are Eq. 21 for sand beds and cohesive banks, and Eq. 22 for coarse non-cohesive materials.

Equations were not developed for the other two curves because these trends are based on rather limited information.

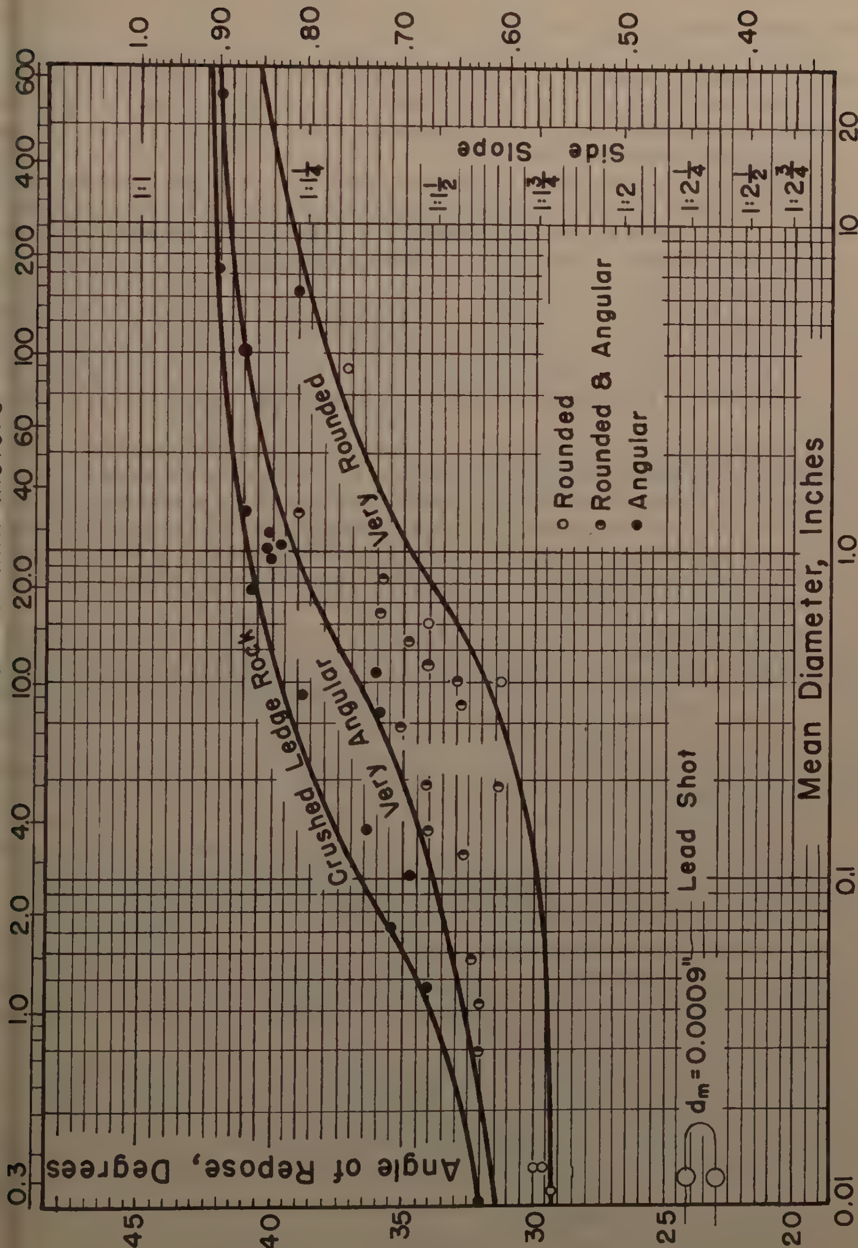


FIG. 12.—ANGLE OF REPOSE OF NON-COHESIVE MATERIAL

Rate of discharge could be correlated with average depth or bed depth, if the need developed. However, the writers feel that hydraulic radius is a more meaningful measure for design purposes.

*Estimating D When R is Known or Vice Versa.*—Corresponding values of D and R are closely related to one another in stable canals, as illustrated in Fig. 1. In this case, D is the average bed depth. A similar correlation could be established relating average depth to hydraulic radius or average depth could be correlated directly with bed depth.

*Selection of A Based on Q and Soil Type.*—The value of A can be determined from the curves of Fig. 6 knowing Q and soil type, or it can be solved directly, if Eqs. 23 or 24 (which depend on soil type) apply. As before, because of limited data, no equation was written for canals with both sand beds and sand banks. Then knowing A and Q, V = Q/A.

The area required to transport a given discharge is maximum for sand banks and beds, somewhat less for slightly cohesive to cohesive banks inclusive and a minimum for coarse non-cohesive banks and beds. The explanation of the preceding is primarily a difference in stability of the different bank materials.

*Determination of Design Slope.*—As shown in the preceding paragraph, the average velocity can be estimated from Q, soil type, and A. Knowing V and R, it is possible to evaluate slope by referring to Fig. 8 which correlates V and $R^2$ S. That is, for coarse non-cohesive materials Eq. 26 is applicable; for canals with sand beds and cohesive banks Eq. 27 applies (Eq. 27 is the same as Eq. 25 suggested by Lacey); and Eq. 28 is used for canals with sand beds and banks.

*Slope Determination by the Blench-King Slope Formula.*—The Blench-King slope formula, when applied in dimensionless form to the Simons and Bender and the India canal data, yielded Fig. 9. The correlation is quite significant for values of $\frac{V\,W}{\nu} = R_e < 8{,}000{,}000$, particularly for the India data. Beyond this point it seems logical to use some other method to estimate S—since even the India data scatter badly when $R_e > 2 \times 10^7$.

To evaluate slope, first estimate W, D, and V. Next compute the parameter $V\,W/\nu$ using an appropriate value for the viscosity. Then enter Fig. 9 and obtain a value for $V^2/g\,D\,S$. Knowing V and D, the slope can be determined.

*Determination of S by the Tractive-Force Method.*—The scope of the original d versus $\tau$ relation (14) has been broadened to include conditions encountered in the canals investigated by Simons and Bender and the India canals (Fig. 11). Using this family of curves an estimate of S can be made by first estimating mean size of bed material, hydraulic radius, type of bank conditions that will result, and by knowing Q.

The major limitations of this method are lack of knowledge of size of bed material and the scatter occurring in the plot—which is, of course, related to the accuracy with which slope can be estimated.

*Summary.*—A summary of the range of the more important variables utilized to establish the relationships which have been presented follows:

Q varies from 5 to 9,000 cfs
Slope varies from 0.000058 to 0.000388
Average width varies from 2 ft to 264 ft
Depth varies from 2.8 ft to 10.5 ft

Sediment concentration varies from 50 ppm to 500 ppm excluding four canals which have concentrations ranging from 2,500 to 8,000 ppm the larger part of which must be wash load.

In conclusion, it may be stated that within the limits of the canal data presented:

1. Both P and W can be estimated with fair accuracy by using Figs. 3 and 2 and/or the corresponding equations.
2. Both D and R can be estimated with fair accuracy by means of Figs. 1 and 5 and/or the corresponding equations.
3. A reasonable estimate of design slope can be obtained by using one or more of the following:
   a. Fig. 8 correlating V and $R^2$ S.
   b. Fig. 9 representing the Blench-King regime equation in dimensionless form.
   c. Fig. 11 relating tractive force, mean diameter of bed material, type of bank material, and discharge.

The relative merits of the respective methods available to help estimate design slope can be comprehended and appreciated more fully by referring to an actual design problem.

## SUMMARY AND CONCLUSIONS

Two basically different theories are currently recognized because of their superiority over other available existing methods used to approximate the design of stable channels. These concepts are:

1. The regime theory of India as developed by Kennedy, Lindley, Lacey, Bose, Blench, and others, and
2. The limiting-tractive-force theory as proposed by Lane and others.

The applicability of the regime theory has been emphasized in this paper.

An investigation of the regime theory verifies that the regime equations of India are only valid for the limited range of conditions upon which they are based as follows:

a. Channels having sand beds, and slightly cohesive to cohesive banks, the banks of which are usually formed by the berming action of the suspended sediment.

b. Channels that are not required to carry a heavy charge of sediment for sustained periods of time. That is, the canals yielding the data upon which the India regime theory is based have their magnitude of charge controlled by sediment exclusion and/or ejection structures so that it is, generally, less than 500 ppm, which is about the upper limit of sediment load (sand and silt size) that can be transported without appreciable bank erosion.

The range of conditions to which this theory applies has been expanded as a result of this study so that canals in each of the following groups can be designed by this method.

(1). Canals formed in coarse non-cohesive material of the type studied by the USBR (14) (charge $<$500 ppm).

(2). Canals formed in sandy material with sand beds and banks (charge $<$ 500 ppm).

(3). Canals possessing sand beds and slightly cohesive to cohesive banks (Good results when charge $<$500 ppm, qualitative results when charge $>$ 500 ppm).

(4). Canals having cohesive beds and banks (charge $<$500 ppm).

Within each of the preceding four classifications it is possible to evaluate area, average width, top width, bed depth, average depth, hydraulic radius, wetted perimeter, average velocity, and side slopes with ease and a practical degree of accuracy. This is accomplished by means of the regime-type correlations involving these parameters, as previously discussed (Figs. 1 to 11).

With the exception of Fig. 8, none of the slope equations or relations investigated reflect the effect of sediment load on slope. The major reason why the Simons and Bender data, as well as other data, have not cast more light on this problem is that the sand-silt sediment load measured in these canals proved to be fairly constant varying only within very narrow limits (concentration $<$ 500 ppm)—thereby making it virtually impossible to determine any meaningful effect.

Utilizing all of the basic data, an attempt to extend the range of applicability of the tractive force method of determining slope to include all types of canals was investigated. The results of this study are fairly well summarized in Fig. 11. In terms of this it is immediately obvious that canals formed in materials in the sand range and finer constitutes a group within which conditions are significantly more complex than in the coarse non-cohesive range. Note that five different curves have been drawn, each representative of a different sub-group of canals, and that Q increases moving from left to right along any curve. Estimating a design slope in the range of sand and finer by means of these curves is in no sense a precise approach, but the figure is extremely useful in that within limits slopes can be estimated, and the presentation gives an insight to the complexity of conditions within this range of operation heretofore unrecognized by the tractive-force approach.

Summarizing, the tractive-force method of design seems to be valid for the coarse non-cohesive range. However, it is recommended that equations of the regime type relating such terms as W and Q and D and Q, should be used to estimate width-to-depth ratio in preference to arbitrarily selecting W and D and then computing a slope consistent with stability of sides and bed, unless working with a coarse material or a stabilization problem where coarse materials will be used to armor plate the channel banks—the natural bank material being of smaller size.

Because of the less significant correlation between tractive force, mean diameter of bed material, etc., as depicted by Fig. 11, within the sand and cohesive range of particle sizes, other relations of the regime type are perhaps equal or superior to the tractive force method for estimating design slope. In any event, it seems logical, based on the validity of regime type area, width, and depth relations, that they should be used to establish W/D regardless of whether regime or tractive force equations are used to estimate design slope.

The disadvantage of imposing an arbitrary W/D ratio on a design problem is that when narrow widths are selected (that is, a small W/D ratio) the magnitude of slope must be limited to avoid bank erosion. This also means a small average velocity and a reduced ability to transport sediment.

The foregoing brings out the advantages of combining the strong points of the two theories, except possibly in those cases where coarse non-cohesive materials are involved. Then apply the tractive force theory directly.

## APPENDIX I.—LIST OF SYMBOLS

| Symbol | Dimensions | Definition or Description |
|---|---|---|
| A | $L^2$ | Area of water cross section |
| b | $L/T^2$ | Bed factor |
| B | L | Bed width of channel |
| C | $L^{1/2}T$ | Chezy coefficient |
| d | L | Median diameter |
| D | L | Average bed depth |
| f | $L/T^2$ | Lacy silt factor |
| n | $L^{1/6}$ | Manning coefficient of roughness |
| $F_r$ | — | Froude number |
| $R_e$ | — | Reynolds number |
| P | L | Wetted perimeter |
| Q | $L^3/T$ | Discharge |
| R | L | Hydraulic radius |
| s | $L^2/T^3$ | Side factor |
| S | — | Slope |
| V | L/T | Average velocity |
| W | L | Average width of channel such that A = WD |
| $W_T$ | L | Top width of channel at the water surface |
| $\gamma$ | $F/L^3$ | Specific weight |
| $\theta$ | degrees | Angle of repose |
| $\nu$ | $L^2/T$ | Kinematic viscosity |
| $\rho$ | $FT^2/L^4$ | Mass density |
| $\tau$ | $F/L^2$ | Tractive force or shear |

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# RESISTANCE TO FLOW IN ALLUVIAL CHANNELS[a]

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## SYNOPSIS

This paper presents the initial results of a flume study of alluvial channels. A detailed classification of the regimes of flow, the forms of bed roughness, and the basic concepts pertaining to resistance to flow are discussed.

## INTRODUCTION

The problem of defining roughness in alluvial channels dates back several centuries. The solution of this problem, at least the complete solution, has thus far eluded man. The principal reasons why only limited answers, which in some cases are of questionable value, have been developed is that the scope of the problem is broad, and that there is a great number of variables which influence resistance to flow in alluvial channels. Roughness in alluvial channels is a more complex problem than roughness in rigid channels because the form of the bed is a function of the flow. That is, not only do the alluvial channel roughness elements resist the flow, but they, in turn, are shaped by the flow. Thus far, most scientists working in this field have been limited by time, facilities, instrumentation needs, and funds. As a result, only small parts of the complete and complex problem have been thoroughly investigated.

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THEORY OF FLOW IN ALLUVIAL CHANNELS

An analysis of flow in alluvial channels is extremely complex because of the many variables involved and the difficulty of measuring them. D. B. Simons, E. V. Richardson and M. L. Albertson discussed[3] the pertinent variables and important dimensionless parameters related to resistance to flow in alluvial channels, utilizing the principle of dimensional analysis.

*Dimensional Analysis.*—The equation relating the selected dependent variable D and the independent variables states that

$$D = \phi\left(B, S_{fc}, S_{fr}, V, S, \rho, \mu, \Delta\gamma, C_f, d, w, \sigma, \Delta\gamma_s\right) \ldots\ldots (1)$$

Using D, V, and $\rho$ as the repeating variables and applying the Pi-theorem

$$\phi_1\left(\frac{B}{D}, S_{fc}, S_{fr}, S, \frac{V\,d\,\rho}{\mu}, \frac{V^2}{\frac{\Delta\gamma\ D}{\rho}}, C_f, \frac{d}{D}, \frac{w}{V}, \sigma, \frac{V^2}{\frac{\Delta\gamma_s\ D}{\rho}}\right) = 0 \ldots\ldots (2)$$

The parameters involving $\mu$ and $\Delta\gamma$ are the Reynolds number $R_e$ and the Froude number $F_r$, respectively. Since $\Delta\gamma/\rho \approx g$ for flow of water in open channels, the Froude number can be expressed in the form

$$F_r = \frac{V}{\sqrt{g\,D}} \ldots\ldots (3)$$

The last term in Eq. 2 can be modified to a drag coefficient $C_D$ for the particles. That is,

$$C_D = \frac{d\,\Delta\gamma_s}{\rho\,w^2} \ldots\ldots (4)$$

Eq. 2 can be rewritten as

$$\phi_2\left(\frac{B}{D}, S_{fc}, S_{fr}, S, R_e, F_r, C_f, \frac{d}{D}, \frac{w}{V}, \sigma, C_D\right) = 0 \ldots\ldots (5)$$

This equation can be simplified by noting that:

1. The width-depth ratio is a term which is probably of secondary importance provided $B/D > 5$. When $B/D < 5$ the effect of side walls may be appreciable.
2. The relative standard deviation $\sigma$ of the size distribution of the bed material undoubtedly influences flow under certain conditions which must ultimately be determined. However, at this stage of the investigation only one sand has been used, eliminating $\sigma$ as a variable.
3. The shape factors for the channel $S_{fc}$ and $S_{fr}$ may be eliminated because the bed is completely alluvial, the cross section is uniformly rectangular except for variation due to bed roughness, and the channel is straight.

Utilizing these simplifications, Eq. 5 reduces to

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[3] "A Study of Resistance to Flow in Alluvial Channels," by D. B. Simons, E. V. Richardson and M. L. Albertson, U. S. Geol. Survey Water Supply Paper 1948A, 1959.

$$\phi_3\left(S, R_e, F_r, C_f, \frac{d}{D}, \frac{w}{V}, C_D\right) = 0 \qquad (6)$$

The parameters in Eq. 6 can be modified to other forms in the same manner that the last term in Eq. 2 was modified to a drag coefficient $C_D$.

## EXPERIMENTAL EQUIPMENT AND PROCEDURE

This study (by the United States Geological Survey (USGS) at the Colorado State University) of roughness in alluvial channels was conducted in a tilting recirculating flume 150 ft long, 8 ft wide, and 2 ft deep (Fig. 1). Any slope between the limits of 0 to 0.013 ft per ft could be set and discharge could be varied from 2 cfs to 21 cfs.

A sand bed 0.7 ft deep was placed in the flume. The sand was a natural river sand obtained from a commercial sand and gravel company located on the Cache La Poudre River. The median fall diameter of the sand, as determined using the visual accumulation size analysis method[4] was 0.45 mm.

The size distribution plotted as a straight line on log-normal probability paper (Hazen, Whipple and Fuller) with a slope of 1.60.

Forty-five runs completed with this sand form the basis of this report. These runs ranged from the condition of no sediment movement to antidune flow. To the writers' knowledge, this is the first set of data, excluding that by G. K. Gilbert,[5] which covers such a broad range of flow conditions.

*General Procedure.*—The general procedure followed for each run involved recirculating a given discharge of the water-sediment mixture in the flume at a given slope until equilibrium conditions were established.

Slope selection was accomplished in a general sense. In any flow system where discharge, depth, and slope can be varied, only two of the three variables can be considered as independent. In a natural stream the discharge and slope are normally independent with depth dependent. In the flume the discharge was independent, slope was independent within limits, and depth was dependent. The slope was preset at the beginning of a run by adjusting the tail gate. Such adjustment indirectly influenced the depth as a dependent variable. Generally, and especially at the flatter slopes (0.0014 to 0.006), the slope of the water surface was adjusted parallel with the bed. With the development of bed configuration, the slope and depth adjusted to the new condition of bed roughness. Thus, for these experiments the slope and depth are a function of Q and the roughness which develops for that regime of flow. The non-uniformity of flow caused by a change in bed roughness was eliminated by continuing the run until the bed slope and the slope of the water surface again became parallel to each other by natural adjustment of the sand bed.

In the tranquil flow regime the slope of each run was adjusted at the beginning by first setting an M1 backwater curve, and then by alternately adjusting the tail gate and measuring the water surface slope until it reached the desired value. For the flat slopes (0.00014 to 0.0006), the bed was carefully screeded to the desired slope before the run. With the steep slopes the bed was only raked to remove the dunes from the previous run before starting the new run

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4 "Visual Accumulation Tube for Size Analysis of Sand," by B. D. Colby and R. P. Christensen, Proceedings, ASCE, Vol. 82, No. HY 3, June, 1956.

5 "Transportation of Debris by Running Water," by G. K. Gilbert, U. S. Geol. Survey Prof. Paper 86, 1914.

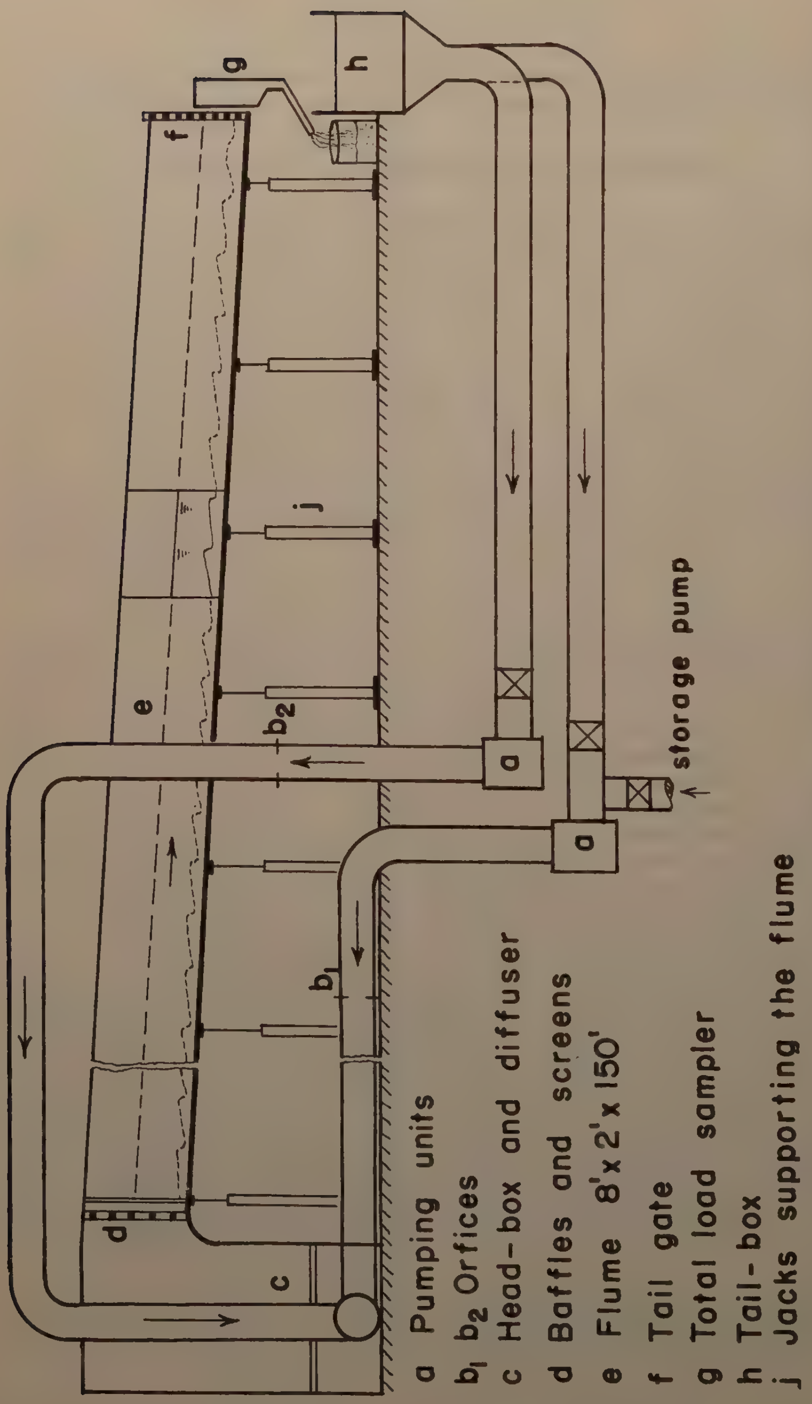


FIG. 1.—SCHEMATIC DIAGRAM OF THE FLUME

because the increased sand movement assisted in obtaining equilibrium conditions quickly. In the rapid-flow regime the slope was changed by altering the bed control at the lower end of the flume and sediment movement quickly established equilibrium conditions.

Equilibrium flow was considered to be established if:

1. The same bed configuration existed over the full length of the flume, excluding the section influenced by entrance conditions.
2. The water surface slope remained essentially constant with respect to time ($\partial S/\partial t = 0$).

The period of time required to establish equilibrium conditions varied with the slope and the discharge. Some runs with flat slopes required 3 and 4 days to achieve equilibrium, whereas, for runs with steep slopes, equilibrium was established in 2 or 3 hr. Every run involved continuous flume operation until it was completed. Whether or not equilibrium conditions were established was based on the measured data and the judgement of the experimenter. To assure the achievement of equilibrium conditions, most of the experiments were continued longer than the required time as indicated by measurements.

After equilibrium conditions were established, water-surface slope, discharge, water temperature, depth, velocity profiles, total sediment concentration, and suspended sediment concentration were measured. Water-surface slope was measured with a Lory point gage and precision level by determining water-surface elevations every 5 ft along the flume. Discharge was measured with calibrated orifice meters. Depth was determined by subtracting mean bed elevation from mean water surface elevation. Velocity profiles were measured at three verticals with a calibrated Prandtl pitot tube. Total sediment concentration was measured by traversing with a width-depth integrating sampler through the nappe at the downstream end of the flume. Suspended sediment concentration was measured with a DH-48 hand sampler. After the run was completed and the flume drained, a profile of the bed configuration was obtained and bed material was sampled. The basic data are presented in Table 1.

## OBSERVED FLOW PHENOMENA

The form of bed roughness in alluvial channels is a function of the sediment characteristics and the characteristics of the flow. That is, the bed configuration can be changed by altering either discharge (which affects the depth), slope, temperature, or the median diameter or size distribution of the bed material. In the flume experiments the bed material was not changed and the slope, temperature, and depth were varied. Under these conditions the following forms of bed roughness were observed.

Tranquil flow regime, $F_r < 1$

1. Plane bed without movement.
2. Ripples.
3. Dunes with ripples superposed.
4. Dunes.
5. Transition from dunes to plane bed.

Rapid flow regime, $F_r > 1$

6. Plane bed and water surface.
7. Standing waves.
8. Antidunes.

TABLE 1.—BASIC VARIABLES FOR 0.45 mm BED MATERIAL

| Run No. | $Sx10^2$, ft/ft | Q, cfs | D, ft | V, fps | T, C | $C_T$, ppm | Form of Bed Roughness |
|---|---|---|---|---|---|---|---|
| 14 | .015 | 3.94 | 0.61 | 0.81 | 10.2 | -- | Plane |
| 17 | .016 | 6.22 | 0.98 | 0.80 | 12.0 | 1 | Ripples |
| 16 | .017 | 5.11 | 0.81 | 0.79 | 12.0 | 2 | Ripples |
| 13 | .019 | 1.84 | 0.35 | 0.65 | 9.0 | -- | Plane |
| 15 | .023 | 5.07 | 0.80 | 0.79 | 11.0 | 1 | Ripples |
| 18 | .031 | 3.62 | 0.58 | 0.78 | 11.3 | 1 | Ripples |
| 2 | .036 | 7.90 | 0.82 | 1.20 | 11.0 | 94 | Ripples |
| 3 | .039 | 7.90 | 0.85 | 1.16 | 11.5 | 101 | Ripples |
| 9 | .040 | 3.84 | 0.55 | 0.88 | 12.0 | 2 | Ripples |
| 1 | .042 | 7.85 | 0.80 | 1.23 | 9.0 | 23 | Ripples |
| 5 | .047 | 7.93 | 0.75 | 1.32 | 11.0 | 26 | Ripples |
| 11 | .049 | 1.95 | 0.35 | 0.70 | 11.5 | 4 | Ripples |
| 4 | .057 | 7.94 | 0.69 | 1.44 | 10.0 | 92 | Dunes |
| 8 | .060 | 3.83 | 0.51 | 0.93 | 12.0 | 8 | Ripples |
| 7 | .078 | 7.98 | 0.70 | 1.43 | 11.5 | 268 | Dunes |
| 10 | .088 | 1.95 | 0.33 | 0.75 | 10.5 | 20 | Ripples |
| 6 | .088 | 3.90 | 0.46 | 1.07 | 9.5 | 42 | Ripples |
| 12 | .106 | 1.95 | 0.29 | 0.85 | 11.7 | 1 | Ripples |
| 19 | .112 | 4.24 | 0.41 | 1.30 | 18.0 | 208 | Dunes |
| 21 | .114 | 12.12 | 0.96 | 1.58 | 16.0 | 641 | Dunes |
| 22 | .124 | 13.54 | 1.00 | 1.70 | 15.7 | 710 | Dunes |
| 25 | .189 | 4.91 | 0.42 | 1.47 | 17.0 | 378 | Dunes |
| 20 | .193 | 8.14 | 0.61 | 1.68 | 16.4 | 508 | Dunes |
| 23 | .247 | 13.34 | 0.65 | 2.57 | 16.0 | 856 | Dunes |
| 24 | .289 | 8.73 | 0.62 | 1.76 | 17.0 | 1200 | Dunes |
| 40 | .301 | 21.41 | 0.81 | 3.32 | 19.0 | 2460 | Dunes |
| 39 | .364 | 20.64 | 0.55 | 4.71 | 19.0 | 3960 | Transition |
| 26 | .366 | 14.45 | 0.34 | 5.38 | 17.0 | 4580 | Transition |
| 28 | .366 | 11.19 | 0.40 | 3.52 | 16.0 | 4230 | Transition |
| 29 | .369 | 4.54 | 0.30 | 1.89 | 17.4 | 1850 | Transition |
| 31 | .432 | 14.85 | 0.44 | 4.24 | 17.5 | 4750 | Transition |
| 27 | .436 | 7.91 | 0.33 | 2.99 | 18.0 | 4100 | Transition |
| 36 | .446 | 3.15 | 0.19 | 2.04 | 19.0 | 1370 | Transition |
| 41 | .466 | 21.62 | 0.54 | 5.05 | 18.7 | 4340 | Transition |
| 30 | .492 | 5.33 | 0.27 | 2.47 | 17.2 | 3550 | Transition |
| 35 | .494 | 5.58 | 0.25 | 2.80 | 17.0 | 4610 | Transition |
| 34 | .546 | 8.44 | 0.28 | 3.73 | 17.5 | 5690 | Transition |
| 33 | .607 | 10.02 | 0.27 | 4.60 | 16.0 | 6810 | Transition |
| 38 | .619 | 21.38 | 0.50 | 5.38 | 19.0 | 6230 | Transition |
| 37 | .620 | 18.87 | 0.43 | 5.54 | 18.5 | 5570 | Transition |
| 32 | .656 | 14.96 | 0.37 | 5.03 | 18.0 | 6180 | Antidune |
| 45 | .862 | 5.58 | 0.28 | 2.50 | 18.9 | 9630 | Antidune |
| 44 | .898 | 10.83 | 0.28 | 4.78 | 19.4 | 15,100 | Antidune |
| 42 | .986 | 13.43 | 0.31 | 5.36 | 20.0 | 11,400 | Antidune |
| 43 | 1.01 | 21.42 | 0.43 | 6.18 | 18.5 | 11,500 | Antidune |

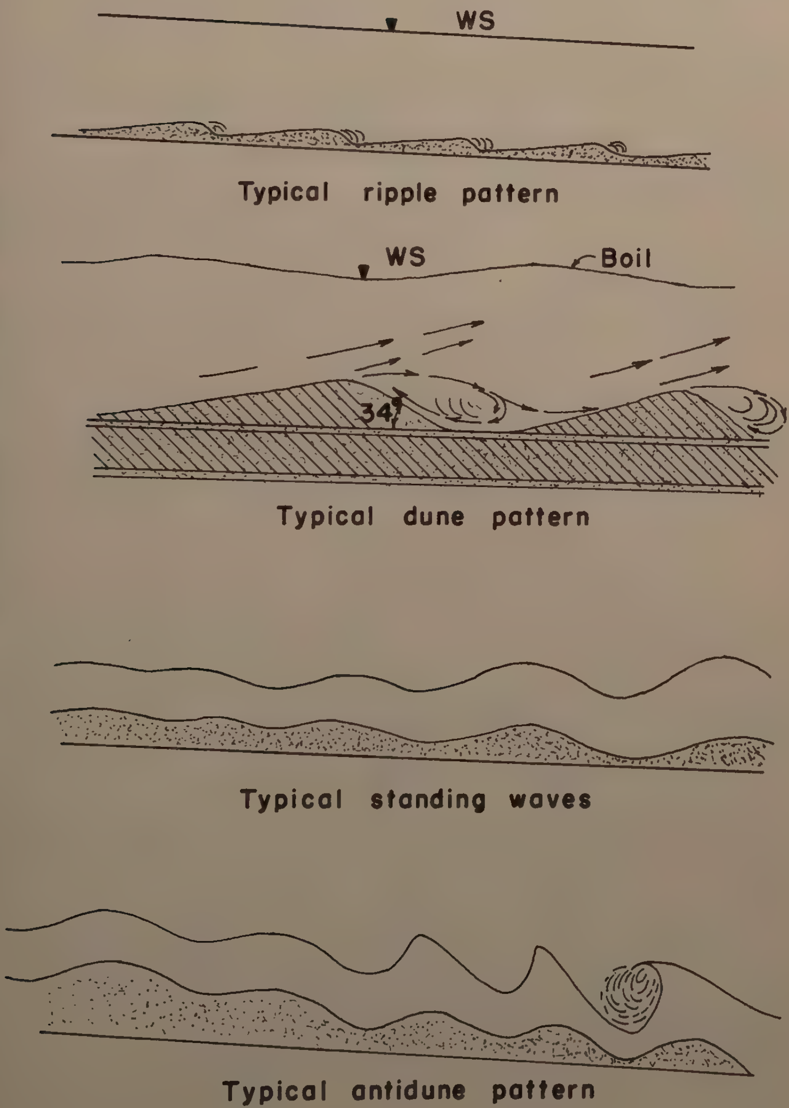


FIG. 2.—MAJOR FORMS OF BED ROUGHNESS

The major forms of bed roughness are sketched in Fig. 2.

Slope and bed material were the dominant factors which determined the form of bed configuration. Although, when slope and bed material remained constant, the form of the bed could be changed by varying depth (changing Q) or changing temperature. However, changing depth or temperature did not change flow from tranquil to rapid unless the slope for a given bed material was close to the critical slope. That is, for a given bed material, a small slope and a dune-bed form of roughness, it was impossible to change to standing waves by changing depth or temperature. With a given depth, temperature, and bed material, all bed forms can be observed by changing slope. This last statement must be qualified because with shallow depths three-dimensional flow and multiple bed roughness forms will occur. It is also possible to have a size of bed material which will not produce some of the bed forms regardless of the depth, slope, or water temperature.

With a given bed material, the change from one bed form to another is not necessarily abrupt. Neither does a particular form occur at the same slope (depth and temperature held constant) nor the same depth (slope and temperature held constant) if the process is reversed. That is, the change from ripples to dunes can occur at a different slope (depth and temperature held constant) than the change from dunes to ripples. This gradual change and/or hysteresis effect results in a transition from one bed form to another. This transition is of major importance when the bed form changes from dunes to plane bed or vice versa. Changes in bed form occur in many natural rivers as a result of a change in discharge. This change will occur if the slope of the energy grade line is close to the critical slope so that a change in Q can cause a change in bed form. For example, the change from dunes at low stage to plane bed at high stage. When this change in bed form occurs there will be a break in the discharge rating curve for that stream. Because there is a hysteresis effect in the transition from dunes to plane bed or plane bed to dunes, the stage where the break in the rating curve occurs depends on whether the stage is rising or falling and also on the rate of change of discharge.

It is important to note that the velocity of the flow in an alluvial channel is not zero at the bed. The velocity of the water through the bed, even though slow, is important, especially when considering seepage force, which is a force or form drag exerted by the liquid on the sand grains within the bed. The magnitude of the seepage force per unit of volume of bed material is equal to the hydraulic gradient of the flow and it acts in the direction of flow. When the magnitude of the seepage force is equal in magnitude and opposite in direction to the submerged weight of a unit volume of bed material (inflow condition), the effective weight of the bed material tends toward zero. Conversely, if the seepage force and submerged weight of bed material act in the same direction (outflow condition), the effective weight of the bed material increased. The direction of flow in the bed can vary from vertically upward to vertically downward depending on bed configuration, direction of flow in the bed, water table condition and regime of flow. The velocity of flow in the bed is largest with the dune-bed form of roughness because the porosity of the material is relatively large due to the relatively loose packing of the bed material resulting from the dune activity, and it is smallest in the rapid flow regime where the bed material is more closely packed. In the flume, as a result of the floor, the velocity of the flow through the bed was probably smaller than in most natural streams with deep sand beds; consequently, the seepage force was smaller. In a natural stream the velocity has vertical components because of inflow to, or

outflow from the bed. The rate of inflow or outflow, which depends on the position of the water table of the surrounding area, can cause large seepage forces. These forces may have a considerable local effect on the form of bed roughness and transport of sediment.

*Tranquil Flow Regime.*

Plane Bed Without Movement.—With the plane bed, and no movement of bed material, the bed was soft and easily disturbed. The word plane is used to emphasize that the bed was not hydraulically smooth. For a hydraulically smooth rigid bed $d/\sigma'$ (the ratio of median diameter of bed material to the thickness of the laminar sublayer) must be less than 0.25. The minimum $d/\sigma'$ for the flume runs was 0.40. Obviously, it is possible to obtain a hydraulically smooth boundary without bed movement by decreasing the slope or the depth. However, a smooth bed or a plane bed without sediment movement in itself has little practical significance. The important factor to consider is the shear at which movement of bed material will begin as the slope or depth or both are increased. It was interesting to note that the shear at which movement began with this bed material was not the shear at which the hydraulically smooth boundary became hydraulically rough. That is, the thickness of the laminar sublayer decreased beyond the shear corresponding to the transition before movement began. It must be remembered that the bed material was nonuniform and computations were based on a median diameter of the material.

Movement of bed material began with a slope of 0.00015 when the depth was increased (by increasing Q) from 0.61 to 0.71 ft the $(d/\sigma')$ - value at the beginning of movement of the bed material was between 0.48 and 0.53. The Manning n for no bed material movement was approximately 0.015, and $C/\sqrt{g}$ varied between 14.0 and 15.0.

When $d/\sigma'$ approached 0.53 movement of the material began and ripples started to form. The ripples first formed at the upstream end of the flume and at any small depression or ridge on the bed, and then continued to develop in a downstream direction from these points of minor disturbance until the bed was covered. As ripples formed on the bed, the slope and depth in the flume increased from 0.00015 and 0.71 to 0.00023 and 0.80 respectively. The Manning n increased from 0.015 to 0.022 and $C/\sqrt{g}$ decreased from 15.0 to 10.3.

Ripples started to form when movement began. Other investigators have reported movement with a plane bed without the formation of ripples. Whether this movement without ripples is a physical fact or the result of the experimental procedure and equipment is unknown. Possibly, movement without ripples results if a sorted bed material is used. The material used in these experiments was unsorted. The width-depth ratio also may be important.

Ripples.—The form of the ripples is illustrated in Fig. 2, and a photograph of typical ripple formation is shown in Fig. 3. For all ripple runs the height of the ripples was less than 0.10 ft and their longitudinal spacing was less than 2.0 ft. Their length-height ratio L/h varied from 10 to 20. The ratio of depth of flow to ripple height ranged from 4 to 24, Manning n ranged from 0.019 to 0.027, and $C/\sqrt{g}$ ranged from 7.8 to 12.4. The upper limit of n-values, and the lower limit of $C/\sqrt{g}$, in general, were associated with the shallow depths.

There was little or no suspended sediment movement with the ripple bed form. The water was sufficiently clear so that the bed was visible at all times. The total sediment load ranged from 1 ppm to 101 ppm and the sediment moved in more or less continuous contact with the bed by rolling up over the crest of the ripples and coming to rest on their downstream face. This sediment did not move again until it became exposed in the upstream part of the ripple as

FIG. 3.—VIEW OF RIPPLE CONFIGURATION, LOOKING UPSTREAM. WATER-SURFACE SLOPE S = 0.000403.

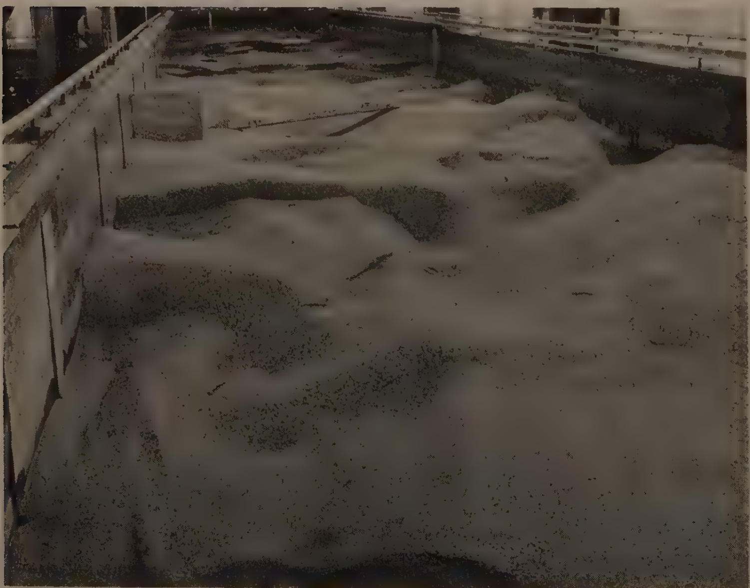

FIG. 4.—VIEW OF DUNE CONFIGURATION, LOOKING UPSTREAM. WATER-SURFACE SLOPE S = 0.00289.

the ripple migrated downstream. Thus, the movement of the sediment particles took place in steps. The length of the step and time interval between steps depended on the length of the ripple and ripple velocity.

With ripples, the bed was soft but not as soft as with dunes. The water surface was very smooth with little visible difference from the water surface for the plane bed without sediment movement. The separation zone downstream from the crest of the ripple caused little jet impingement on the downstream ripple. The reverse flow in the trough moved only the finest particles, and the turbulence was small at the interface between the main current and the separation zone.

When the slope and/or depth were increased beyond a certain limit the ripples were modified to dunes. Because of the difference in relative roughness and resistance to flow it is necessary to treat the two forms of bed roughness separately. Compare the ripples shown in Fig. 3 with the dunes in Fig. 4.

The bed changed from the ripple pattern to a dune pattern when the slope and/or depth were changed such that $d/\sigma'$ was approximately 1.0, and the Froude number was approximately 0.28. The change was abrupt, and the dunes established themselves over the full length of the flume in a few hours. To change from dunes to ripples (Q constant), it was necessary to decrease the slope and increase depth from the values at which ripples changed to dunes. With the reduced transport capacity caused by the change in slope and depth. considerable time was required for the flow to convert completely a dune configuration to ripples. However, ripples superposed themselves on the backs of the dunes in a short time.

Dunes.—The form of dunes is illustrated in Fig. 2, and a photograph of typical dunes is shown in Fig. 4. Considering all dune runs, the dunes ranged in average height from 0.15 ft to 0.52 ft and in length from 4.0 ft to 7.5 ft. The length to height ratio L/h varied from 14 to 28. The ratio of depth of flow to dune height D/h ranged from 1 to 5. Froude numbers varied from 0.38 to 0.60 fixing the upper limit of the unmodified dune bed form. Manning's n varied from 0.018 to 0.035, although observing the bed, much larger values of Manning n would have been expected (Fig. 4). The reason for small n-values is that with a large flume, as with a natural stream, the dunes form in a pattern that allows part of the flow to sideslip or bypass the dune. Sideslippage or meandering of the flow around the dunes was very obvious when a smaller discharge was run over a dune bed formed by a larger discharge.

Dunes move downstream as a result of sediment toppling over the crest and accumulating on the downstream face of the dune. The larger the amplitude h of the dune, the more sediment is stored in the dune and the smaller the dune velocity for a constant transport rate. Considering a particular run, dunes with small amplitude and with their inherent high velocity overtake the larger dunes. This results in a much larger dune and decreases the dune velocity. Velocity of the dunes varied from 0.02 fpm to 0.70 fpm. The large dune velocities were associated with relatively steep slopes and shallow depths. Conversely, the small dune velocities were associated with relatively flat slopes and deep depths. By knowing the average amplitude and velocity of ripples and dunes, the bed load transport can be estimated.[6]

The (L/h)-ratios for dunes and ripples were similar. The ratios varied from 10 to 24 for ripples and 14 to 28 for dunes. This indicates that there is

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6 "A Study of the Effect of Fine Sediment on Flow in Alluvial Channels," by D. B. Simons and E. V. Richardson, U. S. Geol. Survey Water Supply Paper 1948B, 1959.

a correlation between dune height and dune length for a given sediment. That is, if the amplitude of the dune or ripple increases with a change in slope or depth, the length also increases. The variation in the (L/h)-ratios may be the result of the randomness of the dune configuration—which makes it difficult to obtain true average length and amplitude. Some of the variation in L/h may result from including the separation zone in measured lengths of the dunes. Whereas the true length of the dune may only extend from the end of the separation zone to the next crest downstream, the measured length, as used, was from crest to crest. The magnitude of the separation zone increased as slope increased until the transition region was reached where the dunes began to wash out.

The ratio of depth of flow to dune height (relative roughness) D/h was different for ripples than for dunes. Ripples have a larger (D/h)-ratio than dunes. This parameter may help explain why the n-values are smaller for ripples than for dunes. Also with the ripple and dune-bed forms, the largest n values occurred with the smallest (D/h)-values.

It appears from the flume data which is for essentially two-dimensional flow that maximum dune height is limited by the depth of flow. That is, dunes will only grow until their crests are within a certain relative distance of the water surface. Possibly, this phenomenon does not occur in three-dimensional flow. Also, with a given slope and a continuous increase in depth the dune height may reach some maximum value for that size of sediment and water temperature, and increasing the depth beyond this limiting depth will not increase the dune height. If this is true then the shear at which dune height becomes independent of depth must be determined to predict accurately the resistance to flow in an alluvial channel. This shear may be a function of the width of the channel. If dune height becomes independent of depth, the n-values may decrease with increasing stage. That is, h/D the relative roughness will decrease.

With dunes, the bed was very soft and fluid and there was considerable segregation of the bed material. The surface of the upstream slopes and crests of the dunes, which had a negative slope of approximately $2^\circ$ to $4^\circ$, consisted of the fine and coarse material moving downstream. The slopes of the upstream faces were practically the same as those of the upstream faces of the ripples. Ripples formed on the upstream faces of the dunes at the smaller slopes. At large slopes, the ripples on the dunes were not apparent. The downstream face, which had a positive slope between $31^\circ$ and $39^\circ$, largely consisted of the coarse fraction of the bed material. The coarse fraction was the material which could not be swept into suspension at the crest and avalanched down the fore slope. It was deposition by avalanching which caused the dune bed to be so soft and fluid. The material was deposited by the force of gravity and had relatively large voids. If it had been deposited by the dynamic force of the fluid, it would have been packed into place, with relatively small voids ratio. R. A. Bagnold observed[7] and explained the same phenomena with wind-blown dunes. The troughs of the dunes contained a horizontal layer 1/8 in. to 2 in. thick of the fine fraction of the material. Part of the fines in the trough came from the sediment which was swept over the crest and part came from the lower part of the upstream face of the adjacent downstream dune where the main current overrode the separation zone and impinged on the bed (Fig. 2). As the water-

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[7] "The Physics of Blown Sand and Desert Dunes," by R. A. Bagnold, Methuen and Co., London, 1941, pp. 238-240.

surface slope was increased, with the dune form of bed roughness, the toe of each dune was blasted and eaten away by the high velocity water that impinged on it. The erosion provided a source of additional fine material which was deposited in the trough by the reverse flow in the separation zone.

The slopes of the downstream faces of the dunes, which varied from 31° to 39° and averaged 34°, were clearly defined by lines formed by the dark-colored particles as the dune moved downstream (Fig. 2). The angle of repose in air for dry non-cohesive 1 mm sand (the bed material in the flume was 95% finer than 1 mm) varies from 29° to 32° depending on angularity of the particles.[8] It is probable that the angle of the forward face of the dune was greater than the angle of repose of dry bed material in air as a result of pressure on the fore-plane of the dune in the separation zone and the reverse flow in the trough moving up of the face of the dune. This was verified by the fact that as the velocity of flow was reduced to zero in the flume, the downstream face of the dune slumped to a smaller angle of inclination.

With the dune-bed form of roughness, the water surface was uneven and turbulent and the water surface was lower over the troughs than over the crests of the dunes. This was the result of the acceleration and deceleration of the flow as it contracted over the crest and expanded over the trough. This was illustrated for rigid boundaries by H. Rouse.[9] The degree of roughness (turbulence) of the water surface increased as the bed roughness increased. The roughest water surface observed with dunes occured when there were large water surface boils. The water surface in the boils was approximately 0.1 ft higher than the surrounding water surface. However, the roughness of the water surface was not great enough to affect the accuracy of the water surface slope measurements. The turbulence created at the interface between the main flow and the flow in the separation zone dissipated considerable energy and normally the disturbance caused by the interference was visible at the water surface downstream. With the dune-bed form, the suspended sediment concentration, the intensity of the turbulence, the relative roughness, and the velocity of the reverse flow in the trough increased as the slope increased. Along with the dunes, potholes formed which had a depth equal to the height of the dunes. These potholes, as well as the dunes, caused boils on the water surface that were evident downstream from the dunes. The potholes and boils moved downstream in front of the dunes at the velocity of the dunes. Normally from ten to twenty potholes and boils were evident in the full length of the flume with this type of bed roughness. The pothole appeared to develop as a result of the increased strength of the secondary circulation in the separation zone. The combination of the velocity in the direction of flow and the secondary circulation caused a vertical circulation similar to a whirlwind. This rotating motion scoured out additional material at a point in the separation zone and produced a pothole. The existence of these potholes was indicated by the formation of very strong boils which visibly transported considerable suspended sediment upward to the water surface.

The segregation of material by the formation and movement of dunes means that a large number of bed material samples must be obtained and analyzed before the size distribution can be established with confidence. With the flume

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8 "Theory and Design of Stable Channels in Alluvial Materials," by D. B. Simons, Ph. D. Dissertation, Dept. of Civ. Engrg., Colo. State Univ., Fort Collins, Colo., 1957.

9 "Elementary Mechanics of Fluids," by H. Rouse, John Wiley and Sons, New York, N. Y., 1946, pp. 135 and 139.

material, sixty samples were needed before another sample would not alter the median diameter a significant amount. Segregation should have some value, though, because in the field ($F_r < 0.6$), it might be possible to determine the characteristics of the bed roughness by core sampling the bed.

Transition, Dunes to Standing Waves.—The change from dunes to standing waves was complex, and the form of the bed roughness was erratic. The transition occurred when the depth and/or slope were changed so that $d/\sigma' > 2$ and $0.6 < F_r < 1.0$.

With dunes, the maximum Froude number was 0.60. Runs with a Froude number between 0.6 and 1.0 displayed a multiple roughness, that is, a bed form between dunes and standing waves which consisted of washed-out dunes and sand bars. The washed-out dunes and sand bars were 20 ft to 30 ft in length and had $(L < h)$-ratios between 30 and 50.

It is logical that the upper limit of the Froude number, with the dune-bed form, is considerably less than 1.0. The change in roughness, and consequently, the change in resistance to flow and the dissipation of energy were large when the bed changed from the undulating form to dunes, whereas the change in energy from a change in the Froude number was small when $F_r \approx 1$. Thus, the velocity and depth were changed considerably when the bed form changed and small Froude numbers resulted. Conversely, the change from dunes to rapid flow resulted in smaller loss of energy because of the reduced roughness, which caused larger velocities, small depths, and larger Froude number.

There was a hysteresis effect in the change of bed roughness from dunes to standing waves and back to dunes. The value of slope and/or depth for the change depended on the bed configuration prior to the change. If the bed was covered with dunes, a slope of 0.0035 was perhaps required before the dune configuration was destroyed. If the bed was standing waves, a slope of about 0.0025 was required before dunes were formed. This hysteresis effect may have resulted because the energy changed as the bed form changed from dunes to standing waves or standing waves to dunes; that is, potential energy changed to kinetic energy when the form changed from dunes to standing waves, and kinetic energy changed to potential energy when the form of roughness changed from standing waves to dunes. The changes in energy resulted from the large changes in roughness associated with the changes in bed form. Dunes had n-values which ranged from 0.018 to 0.035, whereas the standing waves had n-values which ranged from 0.010 to 0.015.

The fact that a typical dune pattern did not occur with this bed material, and a Froude number equal to or greater than 0.60 is important, especially if the phenomenon holds true for other bed materials. The bed form for $0.6 < F_r < 1.0$ was washed out dunes with rather large $(L/h)$-values. The Manning n-values were 0.018 and 0.021 respectively; these n-values are smaller than n-values for dunes, but are larger than rapid flow n-values. With dunes, it seems that the magnitude of Manning n is related to both spacing and amplitude of the sand waves.

Some runs had distinctly different types of flow occuring side by side. This was probably because discharges which were too small with respect to slope were used which produced three-dimensional flow. This same phenomena was also observed in the antidune range when the discharge was decreased below a given value. With three-dimensional flow, the n-values ranged from 0.014 to 0.023, whereas with standing waves, the n-values were approximately 0.012.

Three-dimensional flow probably resulted because, in the experiments, discharge and slope were controlled and with the steeper slopes the small

discharges set up conditions favorable for three-dimensional flow. Other experimenters may have experienced this flow condition because of the limited discharge capacity of their flume system which caused large width to depth ratios to develop when slope was increased. With the dune bed form, the smallest discharge used was sufficient to insure two-dimensional flow. Presumably, however, by further decreasing the discharge, three-dimensional flow could develop.

*Rapid Flow Regime.*—With rapid flow, $F_r > 1.0$, three forms of sand bed and water surfaces were observed: plane, standing waves, and antidunes.

Plane Bed and Plane Water Surface.—A completely plane bed and plane water surface, for the full length of the flume was only produced once in the total sequence of runs. It was anticipated from existing literature that this would be the type of bed configuration following dunes. However, for this sand, such was not the case. Based on recent tests in which a fine sand was used, it is apparent that the development of a plane bed is intimately related to size of bed material. With fine sand as a bed material plane bed runs are a common phenomenon, and they develop at $F_r < 1$ in the tranquil flow regime.

Standing Waves.—The water surface consisted of symmetrical standing waves of small amplitude. The standing waves formed and gradually disappeared, and unlike antidunes they had no tendency to break or migrate upstream. The forms of bed roughness observed with standing waves, in the order of increasing slope, were:

(a) a diagonal dune pattern cross-laced like a shoestring.
(b) a plane bed.
(c) a symetrical undulating sand wave similar in form to those observed in the antidune regime.

The standing waves formed when Froude number was approximately 1.0; the undulating sand wave bed developed when $1.2 < F_r < 1.3$. The Manning n for standing waves was relatively small, ranging from 0.012 to 0.015. The water surface waves were roughly 1.5 times as high as the corresponding sand waves. Concurrent profiles of the sand bed waves and the water surface waves are illustrated in Fig. 7.

Antidunes.—When the Froude number, which was computed on the basis of average velocity and average depth, was greater than 1.3, and $d/\sigma'$ was greater than 3.0, antidunes formed.

Antidunes are defined as a train of symmetrical sand waves which are in phase with a corresponding train of symmetrical water surface waves. Both trains of waves move upstream, grow in height, then break (Figs. 5 and 6) causing a cyclical fluctuation of the water surface waves and the sand waves. The waves built up from a plane bed with a plane water surface. They grew and moved upstream until one or two of the waves became unstable and broke. Normally, when one wave broke other waves broke immediately thereafter for a distance of 1 or 2 waves upstream and 4 or 5 waves downstream. That is, depending on discharge and slope, from 1 to 8 waves at a spacing of 1 ft to 6 ft usually broke within a short time interval. The chain reaction that followed the breaking of the first wave was apparently triggered by the action of the first wave which broke.

While making concurrent measurements of the bed and water surface in the troughs and crests of antidunes, it was observed that the antidunes became unstable and broke whenever the water surface in the trough of the wave train was approximately the same elevation as the crest of the downstream bed

FIG. 5.—ANTIDUNE WAVE AT POINT OF BREAKING LOOKING UPSTREAM.

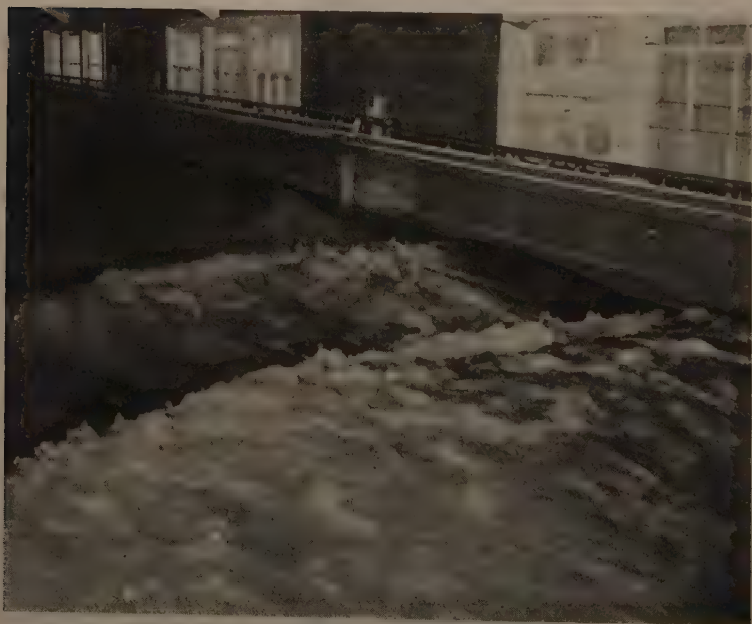

FIG. 6.—ANTIDUNE WAVE AFTER BREAKING.

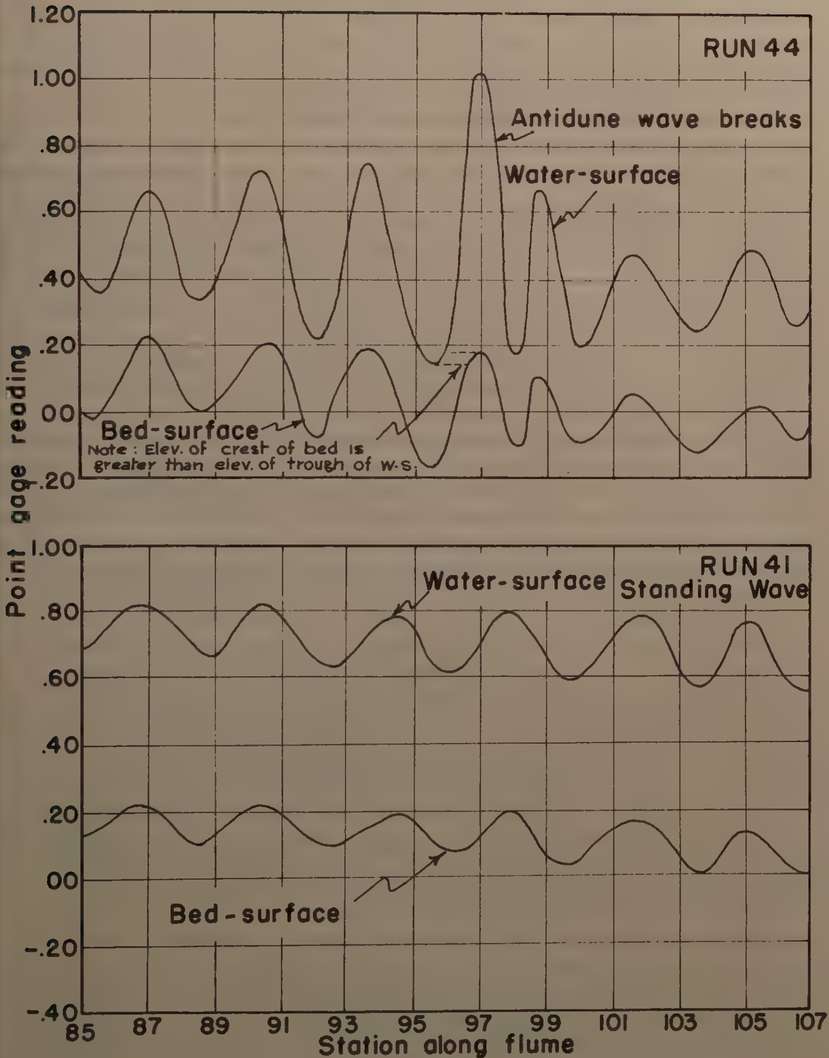


FIG. 7.—THE ACTUAL WATER SURFACE AND BED CONFIGURATION FOR RUNS 44 AND 41

wave (Fig. 7). The measurements also indicated that the water waves were 1.7 times larger in amplitude than the corresponding sand waves. The total height of the water surface waves from trough to crest was from 1.0 to 1.5 times the average depth. When the waves started to break their heights were about twice the average depth. The breaking of the waves was very spectacular. There was considerable turbulence, dissipation of energy, and mixing of the flow. When waves broke they sounded like the surf in the ocean and there is probably some resemblance to the breaking ocean wave.

When the antidunes were building up the bed was very firm and there was no separation between the flow and bed. However, when the antidune broke, the crests of the sand waves became soft and fluid and seemed almost to explode. When the waves broke, the flow was very turbulent, there was separation, and considerable sediment was thrown into suspension. Total load concentrations were very large and ranged from 4,240 ppm to 15,000 ppm. It appeared that the upstream movement of the antidunes resulted from scour on the downstream side of the sand wave and deposition on the upstream side of the wave—consequently, the wave moved upstream.

Except for one run, which was considered three-dimensional, there was only one train of waves in the cross section. The train of waves was located off center at about the left 1/3-point in the cross section. In subsequent runs, however, two trains of waves symmetrically located at the flume were observed several times. The wave trains were not continuous throughout the entire length of the flume. That is, a train of antidunes would either build up, break in a 20 ft to 40 ft length of flume, then build up in another 20 ft to 40 ft reach, or they would build up and break in two or three discontinuous lengths of the flume. The building up and breaking in the different reaches of the flume could be in phase or out of phase.

The period of time it took for one antidune wave to build up and break, and the number of antidune waves which built up at one time, varied with the slope and the discharge. The antidune run with the flattest slope had only one train of 6 to 8 antidune waves which would build up and break at a given time. This train of waves which was about 40 ft long, would perhaps build up in the lower section, then in the middle or upper sections, and again at some other position in a random manne· Normally, about two complete cycles occurred every hour—a cycle includes starting with a plane surface, the building up of waves, the breaking of the waves, and back to a plane surface. For other runs with the steepest slopes there were three or four reaches in the flume in which the train of waves built up and broke both in phase and out of phase, almost continuously.

When the antidunes broke, a considerable amount of water was stored in the flume. Based on observations at the glass-walled section of the flume, when an antidune broke the water in the crest moved upstream, the water close to the bed almost ceased to move until the wave vanished, and shortly thereafter normal flow was restored. With the large slopes, when three or four series of antidune waves in the train would build up and break simultaneously, water was stored in the flume to such an extent that the pumps would surge. In the extreme case, antidune waves would build up and break in phase with the surging of the pumps. This caused the discharge to fluctuate and the antidune activity to increase. Consequently, throughout the full length of the flume so much water was stored in the flume that the tail-water level in the tailbox dropped until the pumps lost their prime. To reduce the surging of the pumps, the level of the water in the sump was raised and excess water was continuously added

to the sump to replace that stored in the flume. When the antidunes were not breaking the surplus of water was discharged through an overflow at the top of the sump.

The storage of water caused by the breaking of the antidune waves probably accounts for the surging discharge that is sometimes observed in alluvial streams with steep slopes. That is, with antidunes water is stored and released in the upper reaches of the stream in a random haphazard manner, but as the flow travels downstream the fluctuation in discharge caused the antidunes to break in a more systematic pattern until surges develop which cause the antidunes to form and break at regular time intervals. Some streams that surge are Muddy Creek in Wyoming and Medano Creek in the Great Sane Dune National Monument, Colo.

Antidune flow using this bed material, hydraulically speaking, was very efficient. The Manning n ranged from 0.010 to 0.013. Even though considerable energy was dissipated when the antidunes broke, the period of time and length of flume over which the antidunes broke was small in comparison with the total time and total flume length. This explains the small n-values. It was observed that, as slope was increased antidune activity increased, the dissipation of energy increased, the discharge coefficient $C/\sqrt{g}$ decreased, and Manning n increased.

Surface velocities, obtained by timing floats along a center line of a train of antidune waves (when they were not breaking) and along a nearby parallel line where the water surface was smooth, showed that the surface velocity was greater in the train of waves. The velocity of flow in the trough of the antidunes was considerably greater than that in the crest. It is possible that the antidunes break when the velocity decreases and depth increases over the crest because the wave height increases until a Froude number less than 1.0 results.

## ANALYSIS OF DATA

It should be borne in mind that the relations presented are based on only one group of flume data, and that the size and gradation of bed material remained constant except as altered by miscellaneous sorting which may or may not have been significant. That is, d and $\sigma$ were constant and w varied only because of temperature changes.

*Variation of Velocity With Depth.*—The variation of velocity with depth and interrelationships of these two variables with the six major forms of bed roughness are given in Fig. 8. Lines of equal Froude number have been superposed on the figure to illustrate the wide scope of the data. Velocities range from those for no movement of bed material to those far in excess of the critical velocity, from 0.65 fps to 6.2 fps. Note that the transition roughness includes those runs in which bed roughness was somewhere between dunes and the roughnesses associated with rapid flow (washed out dunes or transition). Data representing runs of this type plot just below the critical velocity curve. The lines dividing the plot into forms of bed roughness are based on the observed roughness and the Froude number.

*Variation of Resistance to Flow.*—Considering the complete range of conditions investigated the Manning n ranged from 0.008 to 0.035. These n-values have been corrected for side wall effect in accordance with the procedure presented by H. A. Einstein.[10] The magnitude of n increased as the slope in-

[10] "River Channel Roughness," by H. A. Einstein and N. L. Barbarossa, Proceedings-Separate No. 78, ASCE, Vol. 77, July, 1951.

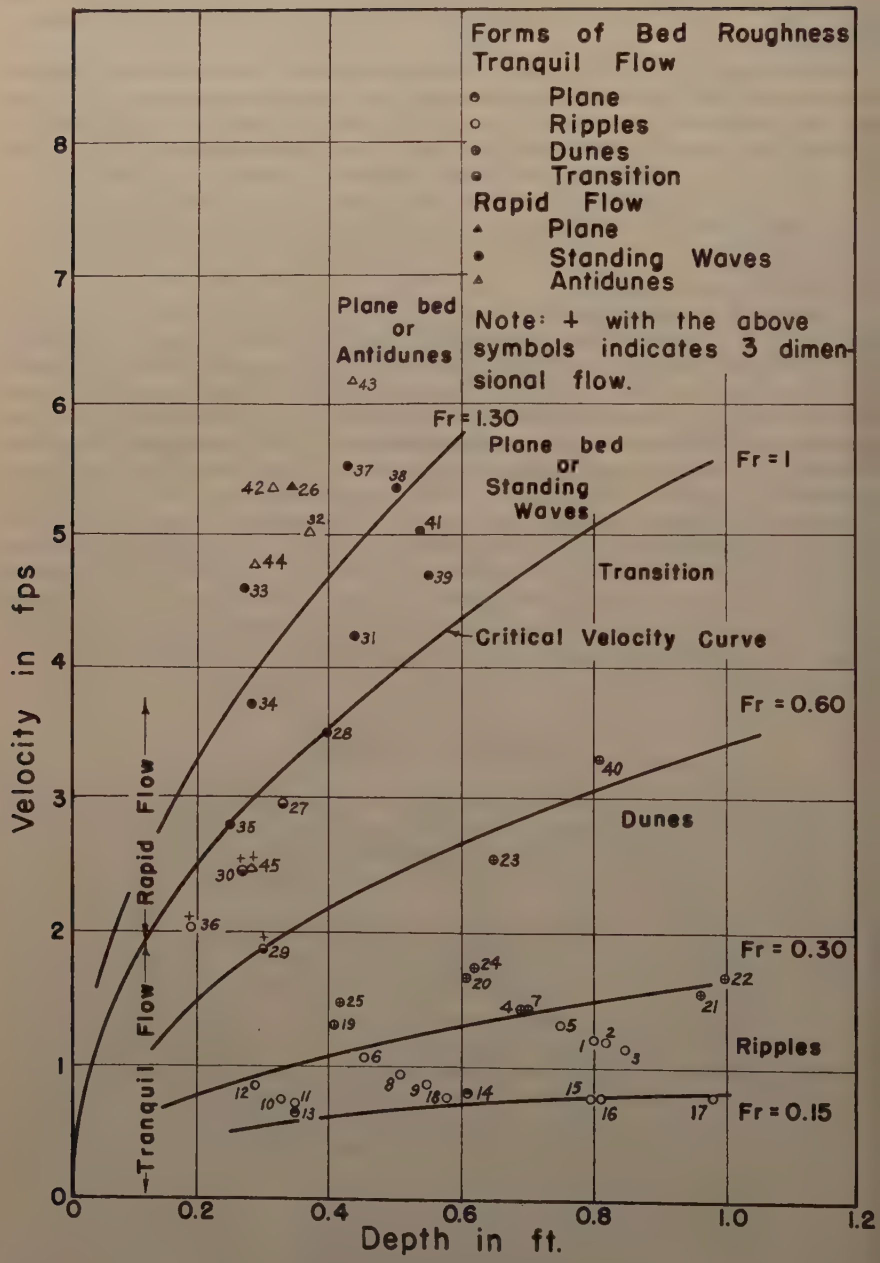


FIG. 8.—VARIATION OF VELOCITY V WITH DEPTH D

creased and the depth remained constant, until the regime of flow shifted from the tranquil regime to the rapid regime; with rapid flow n-values decreased in magnitude quite suddenly to approximately 0.012.

In the tranquil flow regime the Manning n was 0.015 for the plane bed and no bed material movement, ranged from 0.019 to 0.027 for ripples, and from 0.020 to 0.035 for dunes. In the transition from tranquil flow to rapid flow the resistance to flow was also in transition. It shifted with small changes in depth and slope from a large resistance, which was slightly smaller than the resistance for dunes to a small resistance, which was slightly larger than the resistance for rapid flow. With dunes, the magnitude of the resistance to flow is dependent on the spacing and amplitude of the dunes.

In the rapid flow regime, Manning n ranged from 0.0078 to 0.015. The fact that resistance to flow is much smaller than for dunes can be attributed to the change in bed roughness. Dunes have a large separation zone with large form drag, whereas the symmetrical sand waves had little or no separation zone, only the form drag of the particles. Possibly some of the decrease in Manning n can be attributed to the movement of large quantities of sediment as bed load and not entirely to the elimination of the dunes. In the rapid flow regime insufficient turbulence was created to hold large quantities of sediment in suspension. Consequently, the sediment load which was carried in suspension with dunes was carried near the bed. This large concentration near the bed, in turn, markedly changed the properties of the fluid-sediment mixture. The fluid was not homogeneous and a sort of stratified flow resulted which inhibited mixing and reduced the effective bed roughness to extremely small values. The flow had some similarity to plug flow in pipes. During the single run for which n = 0.0078, the bed was plane and the resistance caused by the standing waves was not effective. Under these conditions the plug-flow phenomenon was even more pronounced.

*Variation of Tractive Force and Total Sediment Load.*—Fig. 9 was obtained by plotting the tractive $\tau_0$, which has been calculated by

$$\tau_0 = \gamma \, R \, S \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (7)$$

against the total sediment load concentration. Although there is appreciable scatter, a trend definitely exists. The magnitude of total load $C_T$ increases as the tractive force $\tau_0$ increases. In Fig. 9 each run has been symbolized in accordance with its form of bed roughness. There is a discontinuity between the tranquil-flow data and the rapid-flow data because of the relatively large tractive force associated with runs with large dunes.

The deviation of the runs with dunes from the general trend resulted from an increase in shear $\tau_0$, due to the extreme size of the dunes, without a corresponding increase in concentration $C_T$. It is significant to note that a sudden decrease in $\tau_0$ occurred at about $C_T$ = 2,000 ppm as the bed changed from dunes to plane bed or standing waves. This was caused by a reduction in roughness, which also decreased the ability of the flow to transport sediment. Consequently, $C_T$ was cut in half from tun 40 to run 36, beyond which point $C_T$, in the transition regime steadily increased. No discontinuity occurred at the change from ripples to dunes or from standing waves to antidunes. There was, however, considerable scatter in the rapid flow regime, which shows that other variables are needed to define the phenomenon more completely.

*Various Relationships.*—The relationship between $V_* \, d/\nu$, $(V \, \tau_0)/(V_* \, \Delta\gamma_s \, d)$ and the form of bed roughness was investigated for both regimes of flow (Fig. 10). It is of importance to note the precision with which the relation

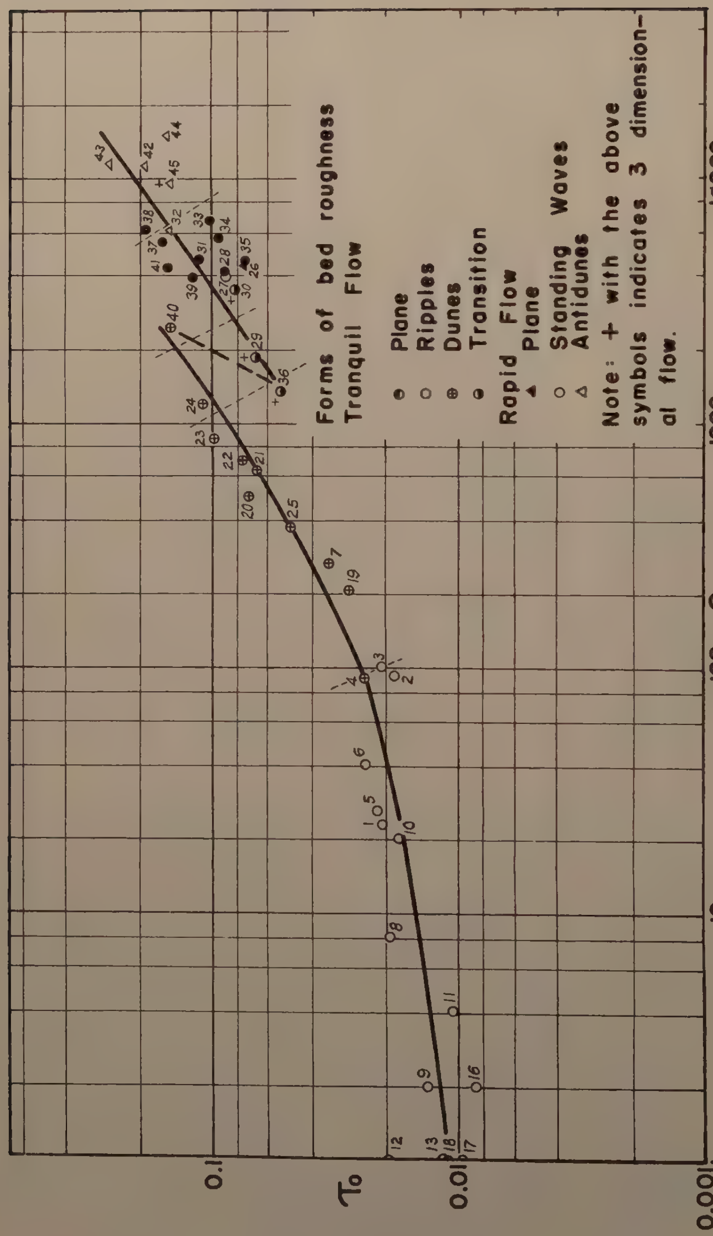


FIG. 9.—VARIATION OF $\tau_0$ AND $C_T$ PPM

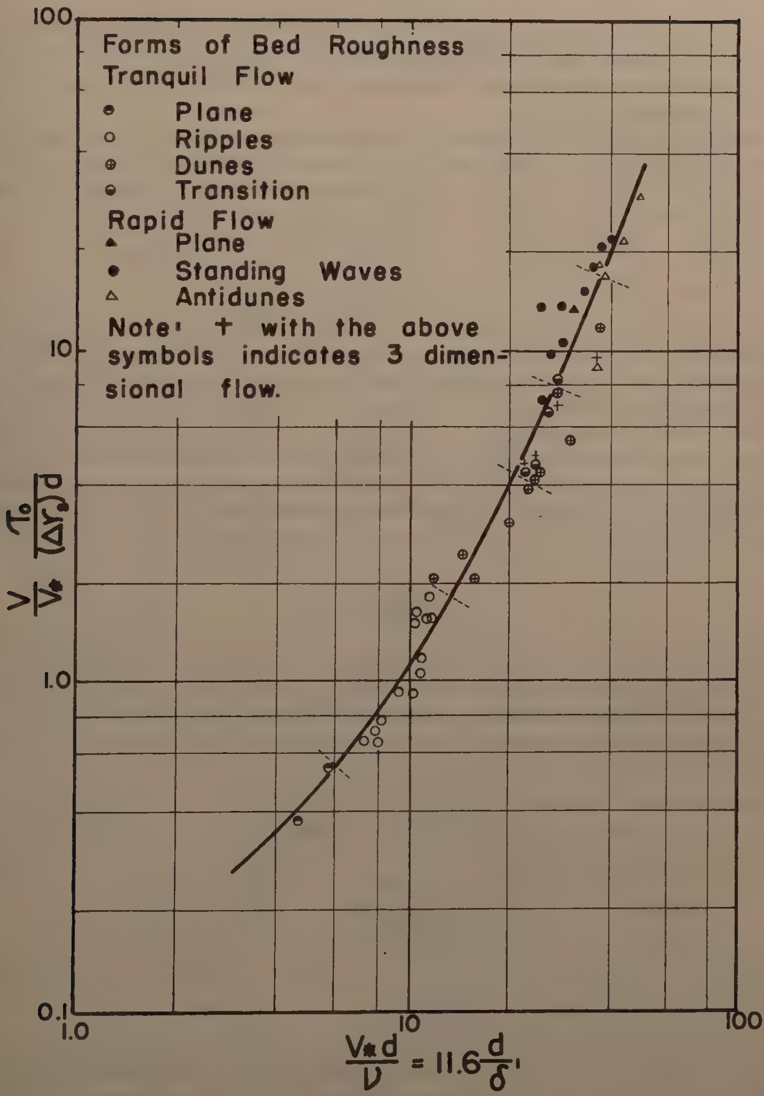


FIG. 10.—VARIATION OF $\frac{V}{V*}\frac{\tau_0}{(\Delta\gamma s)d}$ WITH $\frac{V*d}{\nu}$

$$\frac{V}{V_*}\;\frac{\tau_0}{\Delta\gamma_s\, d} = \frac{\phi(V_*\, d)}{\nu} \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(8)}$$

describes and systematically groups the various forms of bed roughness. The relation can be modified to include the effect of size and gradation of bed material. It is also significant that either or both Fr and $C_T$ can be utilized as the third variable in Fig. 10.

Additional study and adjustment of parameters showed that for the bed material used, $C/\sqrt{g}$ can be computed if the product of a constant and the square of the Froude number is added to the parameter w d $S/\nu$. This is illustrated in Fig. 11, from which

$$V = 0.90 \times 10^{-6} \left[\frac{w\, d\, S}{\nu} \times 10^2 + \frac{5\, V^2}{g\, D}\right] 1.85 \; \frac{\Delta\gamma_s\, S^{-2}}{\gamma} \quad \sqrt{g\, D\, E} \quad \ldots \text{(9)}$$

The form of bed roughness plots very well as a third variable except for minor intermingling of ripple and dune runs at the arbitrarily selected dividing line. It is also possible to utilize total sediment load as a third variable with useful accuracy.

Based on a similar approach, but using $R_e$ or $C_T$ as the additive term instead of $F_r$, figures similar to Fig. 11 which are only slightly less accurate can be obtained. In fact a figure of this type that involves $C_T$ could be used to estimate qualitatively total sediment load in the rapid flow regime.

## CONCLUSIONS

Resistance to flow in alluvial channels varies between wide limits and is extremely comples because the form of the bed roughness is a function of fluid properties, flow and sediment characteristics, and channel geometry. The major forms of bed roughness that were determined from experiments and that are discussed in this report are:

Tranquil flow regime, $F_r < 1$.

1. Plane bed without bed material movement.
2. Ripples.
3. Dunes with ripples superposed.
4. Dunes.
5. Transition from dunes to plane bed.

Rapid flow regime, $F_r > 1$.

1. Plane bed.
2. Symmetrical standing waves.
3. Antidunes.

In the tranquil flow regime the resistance to flow, as measured by the Chez coefficient of discharge $C/\sqrt{g}$, was 14.0 for the plane bed without movement varied from 7.8 to 12.4 for ripples, and from 7.4 to 12.8 for dunes. The larg $C/\sqrt{g}$ values for the ripples or dunes were associated with the deepest depths and hence, the smallest relative roughness.

In the rapid flow regime $C/\sqrt{g}$ varied from 13.9 to 27.0. The largest $C/\sqrt{}$ value occurred with a plane bed and plane water surface. Standing waves, i general, had larger values of $C/\sqrt{g}$ than antidunes. This resulted from the dissipation of energy by the breaking waves. The value of $C/\sqrt{g}$ could be reduce further for antidune flow but for the fact that the period of time and length

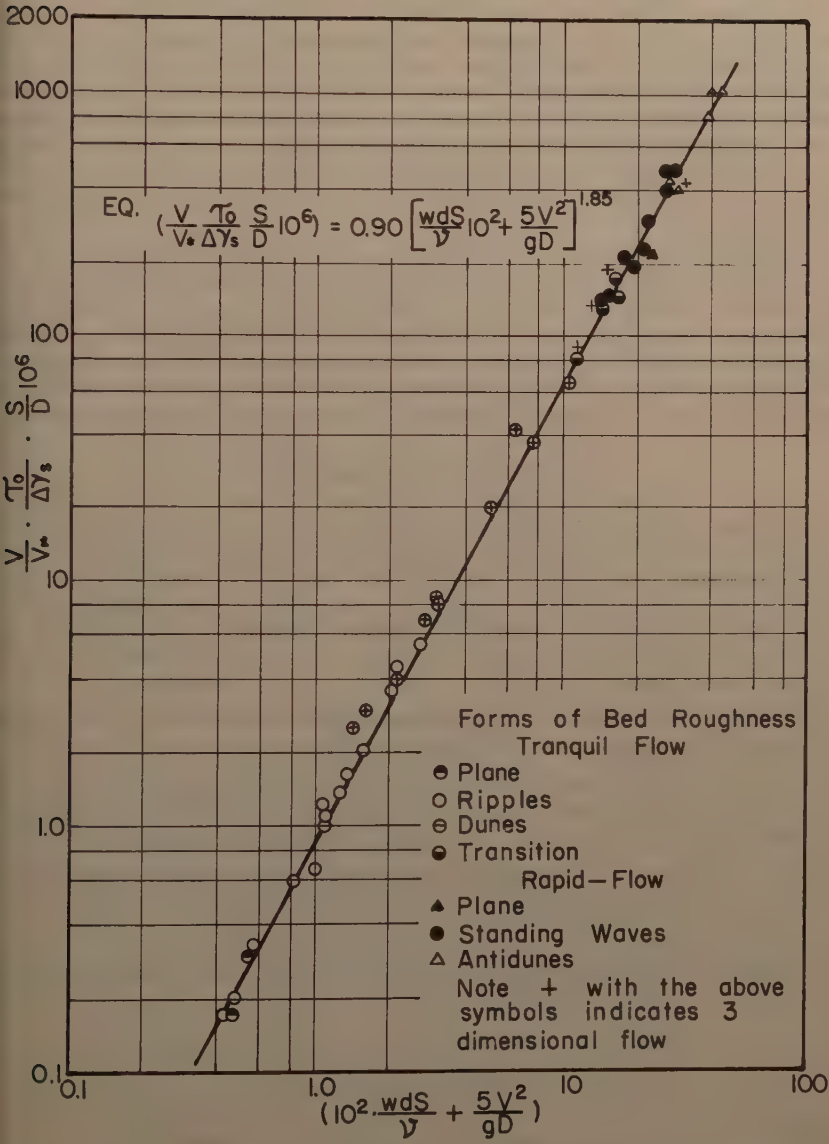


FIG. 11.—VARIATION OF $\left(\frac{V}{V_*}\,\frac{\tau_0}{\Delta\gamma_s}\,\frac{S}{D}\right)10^6$ WITH $\left(\frac{wdS}{\nu}10^2 + \frac{5V^2}{gD}\right)$

flume occupied by breaking waves was small in comparison with the total time and length of flume.

It was observed that with the bed material and the flume used in the experiments, dunes started to wash out and resistance to flow decreased when $F_r > 0.6$. In the transition from dunes to plane bed $C/\sqrt{g}$ varied from 10.0 to 14.1. The magnitude of $C/\sqrt{g}$ increased as the percentage of the bed which was covered with dunes decreased. Also, there was a hysteresis when the dune bed changed to standing waves which depended on the form of the bed prior to the change. That is, with discharge constant, a larger slope was required to change from a dune bed to a standing wave bed than was required to change from standing wave bed to dune bed.

For certain size gradations of bed material, if the slope of the energy grade line is close to critical slope, a change in stage causes dunes to change to plane bed or standing waves or vice versa. This phenomenon occurs in many natural streams and produces a discontinuity in the stage discharge relation. However, because of the hysteresis that is associated with such a change in bed form, the stage at which the discontinuity develops depends on whether the stage is rising or falling and on the rate of change of discharge with time.

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## LIST OF SYMBOLS

| Symbols | | Description | Dimensions | Units |
|---|---|---|---|---|
| $B$ | = | Width of channel | L | ft |
| $C/\sqrt{g}$ | = | Chezy coefficient of discharge in dimensionless form which is equivalent to $V/V_*$. | 0 | -- |
| $C_D$ | = | Drag coefficient for the particle | 0 | -- |

| | | | | |
|---|---|---|---|---|
| $C_T$ | = | Concentration of total load | ppm | 0 |
| $C_f$ | = | Concentration of fine material | ppm | 0 |
| $d$ | = | Median fall diameter of bed material | L | ft |
| $D$ | = | Average depth of flow | L | ft |
| $F_r$ | = | Froude number | 0 | -- |
| $h$ | = | Average height of bed roughness | L | ft |
| $K$ | = | Any constant | 0 | -- |
| $L$ | = | Average spacing of bed roughness | L | ft |
| $n$ | = | Mannings coefficient of roughness | $L^{1/6}$ | $ft^{1/6}$ |
| $n_b$ | = | Mannings coefficient of roughness (Einstein) | $L^{1/6}$ | $ft^{1/6}$ |
| $Q$ | = | Discharge of water-sediment mixture | $L^{3/T}$ | cfs |
| $R_e$ | = | Reynolds number | 0 | -- |
| $S_{fc}$ | = | Shape factor of the channel cross section | 0 | -- |
| $S_{fr}$ | = | Shape factor for the reach of the stream | 0 | -- |
| $S$ | = | Slope of energy gradient equal to water surface slope in steady, uniform flow | 0 | -- |
| $t$ | = | Time | T | sec |
| $V$ | = | Average velocity based on continuity principal | L/T | ft/sec |
| $V_*$ | = | Shear velocity which is $\sqrt{gDS}$, $\sqrt{\tau_c/\rho}$ | L/T | ft/sec |
| $w$ | = | Fall velocity of sediment particles | L/T | ft/sec |
| $\gamma$ | = | Specific weight of water | $F/L^3$ | $lbs/ft^3$ |
| $\gamma_s$ | = | Specific weight of sediment | $F/L^3$ | $lbs/ft^3$ |
| $\Delta\gamma$ | = | Difference between specific weights of air and water | $F/L^3$ | $lbs/ft^3$ |
| $\Delta\gamma_s$ | = | Difference between specific weights of sediment and water | $F/L^3$ | $lbs/ft^3$ |
| $\delta'$ | = | Thickness of laminar sublayer | L | ft |
| $\nu$ | = | Kinematic viscosity | $L^2/T$ | $ft^2/sec$ |
| $\mu$ | = | Dynamic viscosity | $Ft/L^2$ | $lb\text{-}sec/ft^2$ |
| $\rho$ | = | Mass density of water | $Ft^2/L^4$ | $slug/ft^3$ |
| $\rho_s$ | = | Mass density of sediment | $Ft^2/L^4$ | $slug/ft^3$ |
| $\sigma$ | = | Relative standard deviation of the size distribution of the sediment | 0 | -- |
| $\tau_0$ | = | Tractive or shear force developed on the bed, $\gamma$ D S | $F/L^2$ | $lbs/ft^2$ |

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## CONTENTS

### DISCUSSION

# HYDRAULIC ANALYSIS OF SURGE TANKS BY DIGITAL COMPUTER[a]

Discussion by L. Escande
Closure by Nicholas L. Barbarossa

L. ESCANDE.[1]—Mr. Barbarossa's paper constitutes an important contribution to the study of results obtainable, by electronic computation, in the field of certain phenomena involving oscillations of water in surge tanks.

The examples studied relate to a system of two surge tanks arranged in series on the same penstock. The treatment of the problem takes into account a large number of factors which in general are neglected by classic theory, but does not introduce all such factors. Thus, the compressibility of the water and the elasticity of the walls of the pressure conduit are neglected, and by consequence, the corresponding waves of overpressure.

In the case of a simple operation, such as closing or opening, the results obtained can be considered very good. The same cannot be said of the study of stability, for which the results are extremely sensitive to the influence of factors generally considered secondary, and which may lead to results so divergent that they cannot be considered valid.

Mr. Barbarossa concludes, from his studies, that programming for electronic computations should be carried to a greater degree of precision than has yet been attained, and recognizes that the problem has not been solved at this time. It also seems advisable to him to be very judicious in accepting results drawn from normal classic theory, because of the simplifications necessarily introduced into their derivations.

The writer has had occasion to write a paper[2] on analysis of surge-tank phenomena by the method of finite differences, and quite recently we have used electronic computers for the study of certain problems relative to the oscillations set up by successive periodic operations of opening and closing. This study is not yet completed, and thus we cannot draw any formal conclusions. However, we have noted the extreme sensitivity of electronic computations and are not surprised at the divergent results obtained in the study of stability. By definition such a phenomenon corresponds to a limiting case between two domains, and even a minimum influence suffices to orient it toward one or the other of the two. Furthermore, it is hardly possible in programming a study of stability to take into account all the hydraulic phenomena, since the complexities would make the computation too involved for solution.

The writer is in agreement with Mr. Barbarossa on the invaluable assistance which electronic computers lend to the solution of surge-tank problems. For sensitive problems such as those of stability, it is felt that Mr. Barbarossa's

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[a] April, 1959, by Nicholas L. Barbarossa.
[1] Prof., Faculte d'Electrotechnique et d'Hydraulique.
[2] "Methodes nauvelles pour le calcul des chambres d'equilibre," by L. Escande, Dunod, Paris, 1949.

conclusions, concerning the use of electronic computers and the possibilities of practical accomplishment of complete programming, are too speculative. On the contrary, one should not be too critical of the classic methods for study of stability. If they are used with discernment, they may yield results sufficiently accurate to serve practical needs, without introducing an exaggerated margin of safety.

NICHOLAS L. BARBAROSSA,[3] F. ASCE.—It is particularly gratifying to note the number and caliber of the engineers who contributed discussions of the paper. The author takes this opportunity to express his deep appreciation to all who have taken the time to document their thoughts and suggestions on the subject for the benefit of designers and investigators.

Since detailed physical knowledge of the phenomena under discussion is understandably inadequate, there can be no "exact solutions" by computer. As has already been pointed out, the computer is merely a tool, a servant with prodigious capacity for menial work at comparatively small cost. By enlightened exploitation of this capacity, engineers may broaden their understanding of problem phenomena from an analytical standpoint but not, basically, from a physical point of view. Loosely stated, one may say that analytical research can be done in computing centers, but physical research has to be done in the same old places—in the laboratories and in the field. It seems to be rather common to find that each type of research stimulates the other. Thus, it happens, as one example, that computer studies of surge tank analysis described in the paper, based essentially upon conventional methods of long standing, have induced a new look at the whole physical matter of power plant transients. The new look has prompted the same questions regarding physical relationships that must have been asked many times in the past, when mechanical computing centers were made up of only slide rules and desk calculators. Some of these questions have been raised in discussions of the paper. Before attempting to deal with these questions and other points that have been raised, a brief description of pertinent studies recently undertaken by the United States Corps of Engineers Dept. of the Army will be given to demonstrate what the new look by that organization has produced that is of interest and promise to engineers.

The proposal for comprehensive study by digital computer has been accepted as a basis of development of IBM 704 computational programs. These programs are now being developed by the Massachusetts Institute of Technology (M.I.T.) under contract with the Missouri River Division of the Corps of Engineers. Computational programs are to be organized in such a manner as to preserve the general nature of solutions to the highest practicable degree and are to be compartmented in such a way as to facilitate modifications that would be necessary for application of these programs to power plant systems of different but typical geometrical arrangement. Machine programs are to be developed for the Garrison[4] and Oahe power plant systems on the Missouri River and, possibly, for one other undesignated power plant system. A two-year study, terminating sometime in the summer of 1961, has been set up. As reported elsewhere[5] it is planned that the results of this work will be made available to the engineering profession through publication.

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[3] Chief, Hydr. Sect., U. S. Army Engr. Div., Missouri River, Corps of Engrs., Omaha, Neb.

[4] Located about 80 river miles upstream of Bismarck, N. D.

[5] "A New Approach to Hydro Analysis," Engineering News-Record, December 3, 1959.

Since computational programs of power plant transients should be based on actual operational behavior as observed and measured in the field, whenever conditions permit, detailed field tests of the Garrison and Oahe power plant systems have been scheduled by the Corps of Engineers in order that computer results may be evaluated on as sound a basis as practicable and in order to determine to what extent operation corresponds with design. The planning of these tests is being done in close coordination between M.I.T. and several offices of the Corps of Engineers, including the Waterways Experiment Station, Garrison and Omaha Districts, and the Missouri River Division. The first series of field tests will be made at Garrison sometime during the summer of 1960. For each test run, it is planned that both magnetic-tape and oscillograph equipment will be utilized to record in electrical synchronization the instantaneous values of many pertinent variables (pressure and total head at a comparatively large number of points between intake and tailrace, wicket gate opening, turbine speed, power output, and so forth). The response of the many pressure transducers that will be utilized and of other instrumentation will have to be approximately 100 cycles per second in order to keep up satisfactorily with rapid pressure changes during the water hammer phase of a test run. Taken together, the analytical computer studies and directly related operational tests should yield data and design information of an unprecedented nature.

Mr. Jaeger, Messrs. Danel and Ransford, and Messrs. Herak and Rodriguez have all brought up the question of tee losses. The writer concurs that systematic experiments are urgently needed with regard to tee losses under unsteady flow conditions and that computer studies in conjunction therewith could be quite fruitful. Dynamical scale models in which surges are actually produced at the same frequencies as those expected in the prototypes are likely to be expensive. However, the results thus derived may well be the best that can be obtained for particular installations in the design stage. The assurances gained would probably permit closer design of the prototypes, thus achieving economies which may more than offset the expense of the models.

While there is likely to be no serious disagreement as to the great value of well-planned model studies in applicable cases, questions do arise sometimes as to the value of prototype tests for plants which appear to be operating satisfactorily. If these plants are operating satisfactorily today, it is because of the excellent work that has been done in the past. The debt owed to the past can only be discharged by making contributions to the future. Continuing prototype tests, together with applicable model and computer studies, are urgently needed to advance the engineers' over-all knowledge of power plant transient phenomena so that tomorrow's power plants may be designed more economically and may operate even better than they do today.

The original purpose of the IBM 650 program described in the paper was to develop solutions of load rejection, load acceptance, and surge-tank stability problems by machine in essentially the same way that these problems have been solved many times in the past by hand. As demonstrated, the computer handles the first two types of problems satisfactorily but not the third type. Several suggestions have been offered as to how the IBM 650 computer program might be improved to handle the stability cases. Mr. Jaeger proposes the inclusion of the governor equation and Mr. Swiecicki the introduction of speed drop based upon certain assumptions. Despite simplifying assumptions, Mr. Swiecicki's quantitative analysis provides a revealing assessment of the effects of speed drop upon the stability problem. The writer's proposal for the comprehensive IBM 704 program now under development included the

governor equation and speed relationships as well as many others. However, it does not appear feasible to include these relationships in the writer's 650 program as it now stands, for several reasons. First of all, the 650 program is already very large, and major additions thereto might overtax the capabilities of the IBM 650 (with minimum peripheral equipment) but, possibly, not those of a 650 computer with magnetic tape units, indexing registers, etc. Secondly, the IBM 704 program when developed will provide much better answers because it is based upon a minimum number of simplifications. Thirdly, the relative facility of solution characteristics of the existing 650 program, depending, as it does, upon knowing the gate opening G as a function of time, would be lost by inclusion of two more differential equations of general form:

$$\frac{d^3G}{dt^2} + C_1 \frac{d^2G}{dt^2} + C_2 \frac{dG}{dt} = C_3 + C_4\omega + C_5 \frac{d\omega}{dt} \qquad (30)$$

$$P(G,H) - L(t) = I\omega \frac{d\omega}{dt} \qquad (31)$$

In these equations P(G,H) is the power output of the turbine, a function of G and H (and efficiency) or, alternatively, Q and H; L(t) denotes external electrical load, a function of time; I is the familiar $WR_0^2$ divided by g; $\omega$ refers to angular speed of turbine and generator; the C's are the mathematical constants into which the physical constants of the governor system are lumped; and other notation has been previously defined. Eq. 30 is characteristic of the governor systems at the Garrison and Oahe plants. In applicable cases the third order and, possibly, the second order terms might be dropped from Eq. 30.

These remarks are not intended to discourage any further development of machine programs in this field for applications by IBM 650 or comparable computers. Depending upon the objectives and individual judgment, simplifications may be justified, particularly during preliminary stages of design for a particular installation when broad outlines are being established and refinements are not necessary. In such cases at least, machine solutions of the total problem of surge tank stability might be based upon making consistent use of the following: the lumping process suggested by Messrs. Evangelisti and Poggi and implied by Messrs. Danel and Ransford; the analysis of Mr. Swiecicki (based on constant turbine efficiency and other assumptions); the quadratic law for head losses as suggested[6] by Messrs. Oldenburger, Evangelisti, and Poggi; the method upon which the results shown on Fig. 13 are based (discussed later); and other simplifications.

Mr. Sutherland states that it would be "of immense benefit to the profession" to prepare complete details of the writer's program including flow charts and machine codes. The suggested documentation has been accomplished. Mr. Sutherland advises caution regarding the use of turbine performance curves in IBM 704 or 709 applications. The writer is well aware of differences that may occur between expected and actual turbine performance. However, the engineer is confronted with this possibility in any case, whether machine applications are involved or not. Further, obtaining 704 solutions with two or three different sets of turbine performance curves may not be as expensive as

[6] An option was included in the program described in the paper for determining head losses by quadratic relationship, but the results given in the paper are based upon use of the f versus $N_R$ tables. It is not clear why Mr. Oldenburger assumed that the head losses in the paper were taken to be proportional to the "absquare" of the discharge.

imagined, particularly when closer design may result, thus permitting possible construction savings that may more than offset computer costs. Another real prospect is that 704 hourly rates will be drastically reduced in time because of better equipment being available. In fact, this has already happened in some computing centers. Expected turbine performance curves are normally based upon steady state conditions in the model. A vexing but probably unanswerable question is, how well do these curves apply for unsteady flow conditions?

The suggestion by Messrs. Danel and Ransford to test the "error control method used by conducting at least one blank computer run on an artificially-constructed fourth order equation of which the solution is known" is worthy of serious consideration by all who might wish to use the basic method of solution described in the paper. This has not been done by the writer or his colleagues for the reason given in the paper.

While it is true that no independent check for determining computational accuracy exists for non-linear effects, as Messrs. Danel and Ransford point out, the writer believes that one can usually circumvent this difficulty by having the computer print out, for a particular case, all pertinent variables for as many time steps as seem necessary, and then spot checks can be made manually. Table 9 in the paper (as an example) was presented in order to permit such checks.

Following the suggestion of and the method proposed by Messrs. Herak and Rodriguez, additional IBM 650 computer runs based on minor program changes have been made of Test No. 1, Fig. 11, to see whether or not the path of the transient would converge on the new steady point ($M_2$). Part of the results of these runs are presented in Table 10 and in Fig. 13. Up to 1.6 sec, computer results are directly comparable with those of Manual II shown on Fig. 1 of the discussion by Messrs. Herak and Rodriguez. It should be noted that probably Manual Study II does not include the effect of the acceleration head ($h_{a5}$) in the length $L_5$, whereas the computer study does in every case.[7]

In Fig. 13(a) only three of the seventeen loops traced by the computer are depicted for the sake of clarity. Table 10 provides some of the values of $Q_5$ and H that were used to plot the three loops as well as some of the values of other interdependent variables. In following the loops shown in Fig. 13(a) and others not shown, the computer necessarily uses very small time increments. Consequently, it takes about 3 to 4 hr of computer time to obtain results for about 40 sec of problem time. Because of the relatively large amount of computer time required to obtain results for several cycles of surge-tank water surface oscillations (having a quarter cycle of about 70 sec) and because of other working demands on the 650 computer available for this problem, it has been possible within the time permitted for preparation of this discussion to obtain computer results for only about 41 sec of problem time. Hence, a measure of the damping of the hydraulic system over several cycles, as desired by Messrs. Herak and Rodriguez, is not available at this time. Nevertheless, the results shown on Fig. 13(a) are of interest in indicating what can be done by artificial manipulation of gate openings by computer and in showing some convergence toward the new steady state condition (point $M_2$). Under the artificial conditions assumed, the horsepower delivered by the turbine oscillates between a minimum of about 74,000 and a maximum of about 83,000 (78,000 being the

[7] Thus, it is believed that the discrepancy between the Manual II curve and Test No. 1 curve is primarily due to $h_{a5}$ and not to the difference in values of tee loss coefficients. The use of $h_{a5}$ causes Test No. 2 curve to follow a wider path. Compare curves for Test No. 2 and Test No. 3 on Figure 11 for example.

desired new value). At the plant this wide variation would not be permitted, of course, by a well-designed governor system interconnected with a basically stable hydraulic system.

With reference to Fig. 13(b), it will be noted that small-period oscillations

TABLE 10.—OAHE STABILITY STUDIES TEST NO. 1 (CONTINUED)

| t | $Q_1$ | $Q_2$[a] | $Q_3$ | $Q_4$ | $Q_5$ | H | G | $WS_1$ | $WS_2$ |
|---|---|---|---|---|---|---|---|---|---|
| | | | | First Loop | | | | | |
| 0.00 | 5970 | 0 | 5970 | 0 | 5970 | 120.01 | 79.72 | 1537.40 | 1537.36 |
| 0.25 | 5970 | 1 | 5971 | 175 | 6146 | 115.08 | 84.72 | 37.40 | 37.36 |
| 0.45 | 5970 | 14 | 5984 | 356 | 6340 | 114.92 | 88.72 | 37.39 | 37.34 |
| 0.75 | 5970 | 76 | 6046 | 548 | 6594 | 114.74 | 94.72 | 37.39 | 37.31 |
| 1.00 | 5970 | 156 | 6126 | 646 | 6772 | 114.67 | 99.72 | 37.38 | 37.27 |
| 1.05 | 5970 | 173 | 6143 | 658 | 6801 | 115.46 | 100.00 | 37.38 | 37.26 |
| 1.20 | 5970 | 218 | 6188 | 610 | 6798 | 119.73 | 97.00 | 37.37 | 37.23 |
| 1.50 | 5970 | 242 | 6212 | 412 | 6624 | 122.19 | 91.00 | 37.36 | 37.19 |
| 1.80 | 5970 | 212 | 6182 | 194 | 6376 | 123.78 | 85.00 | 37.34 | 37.17 |
| 2.20 | 5971 | 151 | 6122 | -194 | 5928 | 125.74 | 77.00 | 37.32 | 37.17 |
| 2.22 | 5971 | 146 | 6117 | -219 | 5898 | 125.80 | 76.50 | 37.32 | 37.17 |
| 2.27 | 5971 | 135 | 6106 | -268 | 5838 | 123.17 | 76.50 | 34.32 | 37.17 |
| 2.32 | 5971 | 123 | 6094 | -280 | 5814 | 119.42 | 77.50 | 37.31 | 37.18 |
| | | | | Intermediate Loop | | | | | |
| 20.17 | 6006 | 208 | 6214 | 573 | 6787 | 114.86 | 100.00 | 1536.48 | 1536.14 |
| 20.26 | 6006 | 230 | 6236 | 580 | 6816 | 117.56 | 99.00 | 36.48 | 36.13 |
| 20.66 | 6008 | 259 | 6267 | 339 | 6606 | 121.47 | 91.00 | 36.45 | 36.08 |
| 21.06 | 6010 | 217 | 6227 | 28 | 6255 | 123.82 | 83.00 | 36.43 | 36.06 |
| 21.26 | 6010 | 193 | 6203 | -176 | 6027 | 124.57 | 79.00 | 36.42 | 36.07 |
| 21.37 | 6011 | 170 | 6181 | -288 | 5893 | 122.16 | 77.75 | 36.41 | 36.07 |
| 21.42 | 6011 | 158 | 6169 | -300 | 5869 | 118.44 | 78.75 | 36.41 | 36.08 |
| 21.82 | 6012 | 123 | 6135 | 91 | 6226 | 114.20 | 86.75 | 36.39 | 36.09 |
| 22.22 | 6014 | 145 | 6159 | 423 | 6582 | 114.18 | 94.75 | 36.38 | 36.06 |
| 22.42 | 6014 | 183 | 6197 | 528 | 6725 | 114.16 | 98.75 | 36.37 | 36.04 |
| 22.52 | 6015 | 207 | 6222 | 566 | 6788 | 114.93 | 100.00 | 36.37 | 36.02 |
| | | | | Last Loop | | | | | |
| 38.75 | 6095 | 182 | 6277 | 518 | 6795 | 115.22 | 100.00 | 1535.62 | 1535.27 |
| 38.82 | 6096 | 197 | 6293 | 518 | 6811 | 117.37 | 99.00 | 35.61 | 35.26 |
| 39.22 | 6098 | 221 | 6319 | 274 | 6593 | 120.91 | 91.00 | 35.59 | 35.21 |
| 39.62 | 6101 | 188 | 6289 | -50 | 6239 | 123.13 | 83.00 | 35.57 | 35.20 |
| 39.82 | 6102 | 162 | 6264 | -252 | 6012 | 123.90 | 79.00 | 35.56 | 35.21 |
| 39.94 | 6102 | 137 | 6239 | -324 | 5915 | 117.69 | 79.75 | 35.55 | 35.22 |
| 40.04 | 6103 | 117 | 6220 | -250 | 5970 | 114.98 | 81.75 | 35.55 | 35.22 |
| 40.84 | 6108 | 138 | 6246 | 434 | 6680 | 113.75 | 97.75 | 35.53 | 35.20 |
| 40.94 | 6108 | 157 | 6265 | 483 | 6748 | 113.62 | 99.75 | 35.52 | 35.19 |
| 41.04 | 6109 | 177 | 6286 | 510 | 6796 | 115.26 | 100.00 | 35.52 | 35.17 |
| 41.11 | 6110 | 191 | 6301 | 510 | 6811 | 117.36 | 99.00 | 35.52 | 35.16 |
| 41.21 | 6110 | 207 | 6317 | 46 | 6778 | 118.88 | 97.00 | 35.51 | 35.15 |

[a] $Q_2$ Obtained from continuity equation 5.

are imposed upon the falling water surfaces, $WS_1$ and $WS_2$ in the surge tanks, due to cycling of the gate opening, which is changed approximately every 1.2 sec. $WS_1$ and $WS_2$ would probably reach minimum values at about 70 sec and

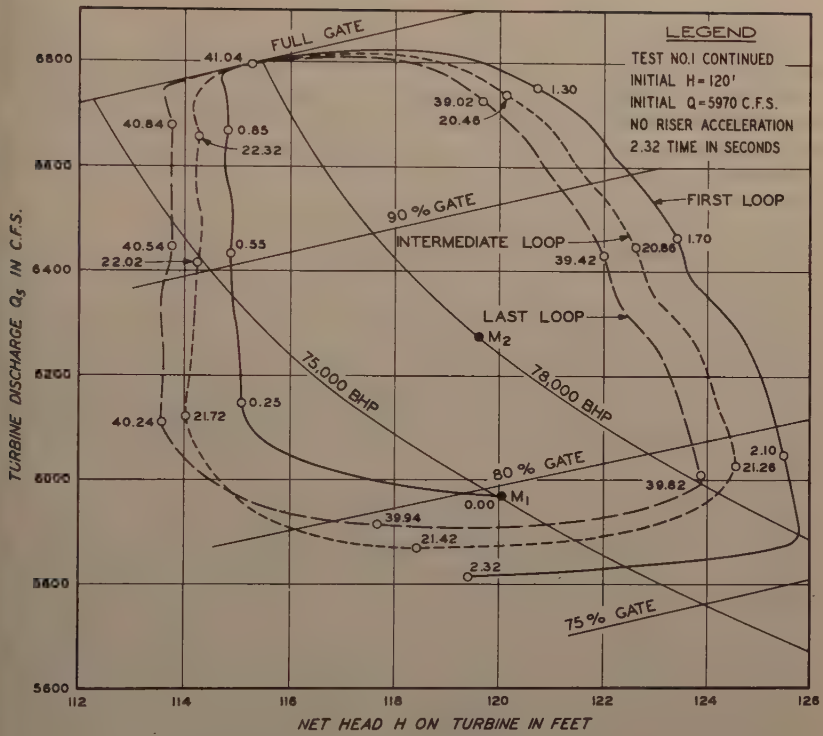


FIG. 13(a).—STABILITY STUDIES

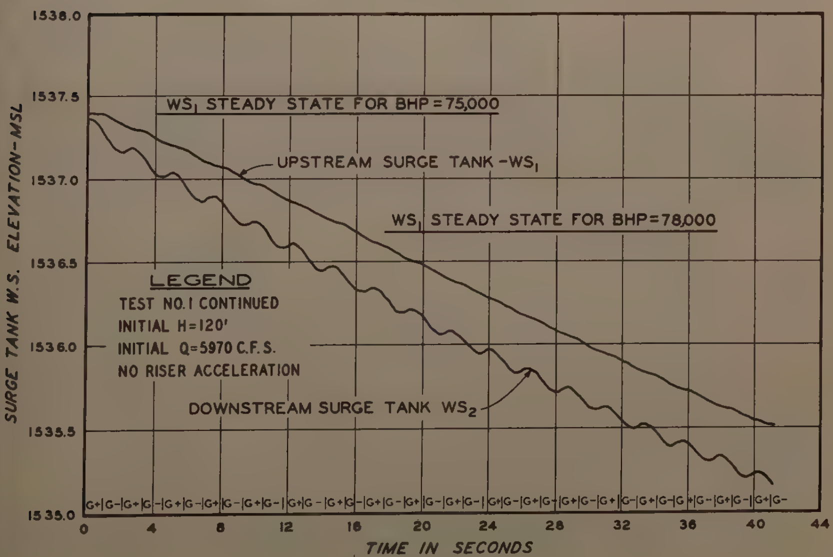


FIG. 13(b)

would then rise, overshoot the new steady levels, and oscillate until dampened to quiescence. $G^+$ indicates that the turbine gates are opening and $G^-$ indicates that they are closing. Although no time lapse is indicated between $G^+$ and $G^-$ on Fig. 13(b), a governor dead time of 0.05 sec was actually used in the computer program, as suggested by Messrs. Herak and Rodriguez. For each $G^+$ and $G^-$ interval, gate movement was assumed to be linear; that is, the gate opening was assumed to change at the rate of 20% per second (five-second governor).

Mr. Swain's general remarks on the use of computers are worthy of note, particularly by engineers who have not yet had an opportunity to be directly associated with computer applications of engineering problems. These remarks substantiate the writer's convictions and experience in this field. With reference to the "table look up" routine, it should be noted once again that specific turbine performance values (Q, H, and G values) during computer runs were obtained in the author's program by double interpolation in the input table (Table 3). This point is emphasized in order to focus attention on the fact that it is not always necessary to enter mathematical relationships into machine programs.

Mr. Escande states, ". . . one should not be too critical of the classic methods for study of stability . . .," whereas Mr. Jaeger congratulates the author "for his criticism of the assumptions on which the present surge stability theory is based." No criticism as such of the classical methods was intended. The author simply desired to illustrate how the known limitations of classical methods might be overcome to a great extent through new approaches, made practicable only recently by the advent of modern computers. The writer cannot agree with Mr. Escande that "the use of electronic computers and the possibilities of practical accomplishment of complete programming are too speculative." Granting that some speculation is involved, as is the case in any new endeavor, should not the new potentialities for substantially better understanding of power plant transient phenomena through comparatively inexpensive computer analyses be fully explored? Is it not necessary, more than ever, to continue both types of research, physical and analytical, utilizing the best tools available?

Mr. Jaeger states that "further research on surge stability is definitely wanted." He suggests that "a comprehensive paper summarizing all these new efforts should analyze the up-to-date position of the problem and show the way to further research." His recent paper,[8] advocating further research, is an excellent review on that subject and is recommended to all who desire to pursue the subject further.

[8] "A Review of Surge-Tank Stability Criteria," Paper No. 59-A-270, ASME, presente at ASME Convention, Nov. 29-Dec. 4, 1959.

# RESISTANCE PROPERTIES OF SEDIMENT-LADEN STREAMS[a]

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Closure by Vito A. Vanoni and George N. Nomicos

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VITO A. VANONI,[1] F. ASCE, and GEORGE N. NOMICOS.[2]—The authors are indebted to the writers who, by contributing discussions which re-examine some of the ideas in the paper, have added greatly to its value.

Messrs. Laursen and Chou have advanced the idea that the resistance to flow offered by grains in the bed is different when they are moving than when they are fixed and suggested that this may account for the lowering of the resistance observed for sediment laden flows. Although such an effect may exist it cannot account for all of the observed change in flow resistance. The fact that the Von Karman k is observed to decrease even for small amounts of suspended load means that there is a substantial internal effect which modifies the turbulence. This effect is always in the direction of reducing the friction factor or the flow resistance.

Mr. Laursen's suggestion that one can study, experimentally, the effect of moving grains on resistance represents an interesting extension of the present work. This raises the whole question of the effect of sediment on flow during cases where the transport rate of sediment is low and most of the material is being carried as bed load. An investigation of this problem would shed further light on the validity of the idea that the principal action of the sediment on the turbulence occurs near the bed where the concentrations are high and where most of the turbulence is generated.

Mr. Chou was correct in inferring that the slope is difficult to measure. As a matter of fact this is one of the most difficult measurements to make both in the field and laboratory and the one that probably has the greatest error in it. In making the experiment great care was taken to avoid end effects which would involve either backwater or drop down curves. When a sample as small as one liter was taken from the flume to determine sediment concentration a change in depth of water could be observed immediately. This was corrected by replacing clear water as the samples were being withdrawn from the flume.

The authors are in agreement with Mr. Chou's general description of turbulence and turbulence production. Turbulence is produced at a given rate on the one hand and dissipated to heat by damping at a given rate on the other hand. When the rates of production and dissipation are equal, steady state has been established as presumably occurred in the flume during the experiments. The intensity and other characteristics of the turbulence in a flow are principal factors in determining the velocity distribution. When sediment is added to a flow and is moved by it, it appears that the turbulence is changed either by modifying its production or its dissipation rate. This also changes the velocity

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[a] May, 1959, by Vito A. Vanoni and George N. Nomicos.

[1] Prof. of Hydr., California Inst. of Technology, Pasadena, Calif.

[2] Specialist-Scientific Research Engr., Republic Aviation Corp., Farmingdale, N. Y.

distribution as has been observed. It is true as Mr. Chou observed that in a flow "turbulence has been created at the expense of the energy stock of the flow." However, once stream energy, that is, potential energy, has been converted to turbulence it is no longer recoverable to produce forward flow of the stream. In view of this the authors do not see how by "dumping" its suspended load a stream can release the energy that is necessary to accelerate the flow as suggested by Mr. Chou. By depositing its load a stream can modify its turbulence and hence velocity but it cannot make available additional energy to produce flow. Mr. Chou states, "Only fine particles less than 0.2 mm can be thrown up into suspension by and move with turbulence generated purely by hydraulic resistance." This is not borne out by observation as much coarser material than this is found in suspension in flumes as well as in natural streams.

Mr. Chou states that velocities are higher on rising than on falling stages of a stream, because the slope is higher during the rising stages and less energy loss is involved. This seems like a reasonable statement, but in large rivers the rates of rise, and hence the previously mentioned effects, are extremely small and one must look for other explanations for the observed phenomena. As an example Carey and Keller (15)[3] report that the duration of the rising part of the hydrograph in the lower Mississippi was about 3 months. In this case it seems reasonable to assume that the effect of the rate of rise is very small and that one can consider the flow as being essentially uniform.

Mr. Chou's suggestion that one way to attack and clarify this problem is through direct measurements of turbulence is certainly a pertinent one. It points up the dire need for instruments for making such measurements which at the present time are only in the very initial phases of development.

Messrs. Simons and Richardson bring up the very interesting and important problem of the effect of the scale of the system on sediment transportation phenomena. This is a problem that workers in sedimentation studies have hardly touched and is one that must be faced if results of laboratory work are to be applied to field conditions. Certainly one of the ways to study the problem is actually to work in larger and larger streams. However, it appears to the writers that considerable understanding of transport problems can also be gained by continuing to study the phenomena involved. Much of this can be done in small flumes which are cheaper to operate and more convenient to work with than large ones. In this way laboratories that do not have large facilities can continue to make significant contributions to this field.

The authors welcome the discussions of bed forms. The terminology used by the authors is that outlined by the subcommittee on sediment terminology[4] of the American Geophysical Union. This committee did not recommend a term to describe a bed without dunes or other forms. The authors used the term "flat" and Simons and Richardson use "plane" to describe such a bed. The authors prefer the term flat over the absolute term plane because according to definition it allows for slight irregularities in surface which, in fact, do occur and hence, is a more precise term. It is apparent that some additional forms have been found by Simons and Richardson and that the problem is not as simple as it first appeared. To the list of bed forms should be added the "sand wave" observed by Brooks (10) to occur between the condition of "dunes" and "flat bed" shown respectively in Fig. 1(c) and Fig. 1(d) of Simons' and Richard-

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[3] Numerals in parenthesis refer to the Bibliography references in the original paper.

[4] Report of the Subcommittee on Sediment Terminology, Transactions, Amer. Geophysical Union, Vol. 28, No. 6, December, 1946, p. 936.

son's discussion. The information reported on the effect of size and grading of the bed material on the form of the bed roughness is most interesting. In experiments reported in the paper the gradation of the two materials used was about the same; however there was a change in the mean size as shown in Table 2. Experimental results reported previously (10,18) (for instance see Table 7 ref. 10) indicate that for flows of the same depth and velocity in a 10 1/2-in. flume which form dunes, the grading of sand does not greatly affect the observed friction factor. Detailed measurements of the dunes were not made in these experiments but photographs show only modest differences in size and shape of dunes.

Simons and Richardson question the validity of ascribing all of the change in friction factor between sediment laden flows and comparable clear flows to the damping effect of the sediment. They would ascribe some of the effect to the change in the intensity of turbulence in the two cases. The authors have used the idea that the significant difference between the two cases, that is, the one with the loose bed and sediment in motion and the other with the fixed bed and clear water, is the sediment load and that therefore the cause of the observed change in roughness must in some way be due to this sediment load. If the turbulence intensity is greater in one case than in the other then the argument would be that this is caused by the sediment in the flow or the lack of it, whichever the case may be.

These writers also refer to their very interesting findings that extremely high concentrations of fine material such as silt and clay have a very important effect on the behavior of a stream. The explanation of these effects must involve one or both of the mechanisms for changing the roughness factor of a stream, that is, modification of the turbulence and of the bed forms.

Simons and Richardson question the contention of the authors that the smaller friction factor of the rising flood can be explained by a difference in bed configurations between the river on rising and falling stages. They are more inclined to explain this difference by a time lag in the change of the bed forms. The authors feel that because during floods the stream actually has a tremendous capacity for transporting sediment and hence changing the bed configuration there would not be a very large lag. This is especially true in the case of the Mississippi River where the times involved are measured in days and weeks and are not only a matter of a few hours. A further reason for not accepting the idea of "time lag" is that the authors are not willing to discard the idea advanced by Brooks that in order to get a single-valued relation for stream characteristics one must consider the sediment load as an independent or imposed variable. According to this concept, a stream would adjust its bed forms, friction factor, mean velocity and stage to enable it to transport the sediment load delivered to it. The bed forms and other hydraulic quantities can be thought of as resulting from the sediment load instead of vice versa. This idea can now be applied to explain the rating curver with a loop in it (15). One can first observe that the sediment load is higher during the rising than during the falling stage. Then according to the above concept the stream will adjust itself to carry the load. To carry the heavier load delivered to it on the rising stage the stream will reduce the friction factor of the bed by modifying the bed form which will result in reducing the stage as observed. The validity of this concept and the results deduced from it as well as the idea of "time lag" in changing bed configurations need further evidence to determine their validity.

# PRESSURE CHANGES AT OPEN JUNCTIONS IN CONDUITS[a]

Closure by William M. Sangster, Horace W. Wood, Ernest T. Smerdon and Herbert G. Bossy

WILLIAM M. SANGSTER,[1] M. ASCE, HORACE W. WOOD,[2] F. ASCE, ERNEST T. SMERDON,[3] A. M. ASCE, and HERBERT G. BOSSY,[4] F. ASCE.—The contributions of the discussers are gratefully acknowledged by the authors. Additional data have thereby been brought to light which tend essentially to corroborate the authors' findings.

Mr. Geyer's comments on the desirability of shaping pipe entrances and exits as well as junction and manhole floors are certainly well-founded on non-hydraulic bases. The authors were, of course, basing their conclusions on the hydraulic aspects of the situation, particularly as applied to storm sewer systems. Extension of the conclusions to include sanitary sewer systems was not intended and certainly is not advocated.

The verification of the authors' results for equal-size pipelines by Messrs. Blaisdell and Manson is gratifying. Comparison of their results[5] with those of the authors does indeed indicate a remarkable agreement.

The answers to the questions raised in the last paragraph of the discussion by Messrs. Blasidell and Manson lie fundamentally in the assumptions inherent in Eq. 4. As stated, it was assumed that the momentum of the flow in the lateral is ineffective in maintaining flow in the downstream pipe. This assumption is obviously not valid if the junction width transverse to the main line is only slightly greater than the diameter of the downstream pipe. Figs. 10 and 11 clearly show that the size of the junction materially influences the magnitude of the loss at the junction. In the smaller sizes the lateral flow is apparently more effective in maintaining the flow in the downstream pipe with the result that a lesser drop in pressure across the junction is required.

Eq. 4 should not be expected to be in close agreement with results obtained with pipes joined directly.[5] Rather Figs. 10 and 11 and the supporting Eqs. 14 and 15 are pertinent to these cases. No fundamental differences between the results presented therein and those in Manson and Blaisdell's work[5] are evident.

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[a] June, 1959, by William M. Sangster, Horace W. Wood, Ernest T. Smerdon and Herbert G. Bossy.

[1] Assoc. Prof. of Civ. Engrg., Univ. of Missouri, Columbia, Mo.

[2] Prof. and Chmn. of Civ. Engrg., Univ. of Missouri, Columbia, Mo.

[3] Assoc. Prof. of Agric. Engrg., Texas A & M College, College Station, Tex.

[4] Highway Research Engr., U. S. Bur. of Pub. Rds., Washington, D. C.

[5] "Energy Losses at Draintile Junctions," by P. W. Manson and F. W. Blaisdell, Agricultural Engineering, Vol. 37, April, 1956, pp. 249-252, 256.

# DETERMINATION OF HYDROLOGIC FREQUENCY FACTORS[a]

## Corrections to Discussion by William H. Sammons

CORRECTIONS TO DISCUSSION BY WILLIAM H. SAMMONS.—In the February Journal of the Hydraulics Division on page 111, for $C_s = 6.80$ under 99 column, 0.58453 should be 0.68453. On page 112, for $C_s = 11.90$ under 99 column, 0.51510 should be 0.52510.

[a] July, 1959, by Ven Te Chow.

# CONSISTENCY IN UNITGRAPHS[a]

Discussion by C. O. Clark

C. O. CLARK,[1] A. M. ASCE.—The author has presented unit hydrographs ınd data for one of the unique streams of the world in the form of a plea for nore consistency in unit hydrographs. The paper illustrates the problems of letermining a unit hydrograph by the originally presented methods, which reuire a flood record for a unit duration storm, when there are no such records, ut only records of much larger multiple duration events.

The results of such application produced a noteworthy unit hydrograph, noteorthy because of its two peaks and the conviction of the author that these eaks are characteristic of the watershed.

A number of years ago, the writer introduced a methodology[2] seeking conistency in derivation, utilizing data not dependent on having a record from a onventionally uniform storm of unit duration. It was also illustrated with hyrographs exhibiting double peaks, both in the records and in the unitgraphs, ikewise attributed and compared to watershed characteristics, particularly atershed shape, length, slope, and factors contributing to the storage and disharge capabilities of the waterway. Perhaps the problems of unusual streams nd these basic concepts of stream flow chronology could be illustrated for ther students by a comparison of derivations and a discussion based on such omparisons.

The author kindly furnished a map, reproduced in Fig. 11, showing the domiant configurations of the stream, Karnafuli River at Rangamati, Pakistan. A lance at the pattern suggests several possible reasons for a two peak unit hyrograph for short duration rainfall. One such reason, watershed shape, is een in the relatively small amount of watershed contributing to the main chanel in the zone from Dimagiri to Barkal and the relatively large contributing reas above and below this zone. This effect would be even more of the proortions indicated by the author if the hydrograph to be considered were at aptai, for in that case the areas above and below the constricted zone would e about equal and the two distinct peaks would be quite reasonable. This conideration may still be applicable, however, because this stream is so unusuly small in slope, about one foot in four miles, that the stage at Rangamati ay indeed be a function of flow farther down stream. In fact, at low stages, is so influenced, in that case by effects from tides at the mouth of the river, any miles farther downstream. But this effect is a question in itself, that ould be quickly answered, if one had measured the flow of the stream a large ımber of times and had determined that the flow or slope was always the same

[a] August, 1959, by Bertram S. Barnes.

[1] Hydr. Engr., Tulsa, Okla.

[2] "Storage and the Unit Hydrograph," by C. O. Clark, Transactions, ASCE, Vol. 110, 45, p. 1419.

for the same stage, or that it was not. One can suspect the latter, but if the stream is truly a unit hydrograph stream, a stream of constant chronological regimen, it would have to be the former. This, of course, is one of the many problems.

For this discussion, the premise will be held that the stream is susceptible to unitgraph concepts of analysis within the range of accuracy desired, as predicated by the author. Thus, it is possible either to deny the downstream effects or to lump them in the many causes for troublesome erratic deviations.

Shape may be a factor in another significant way, too. There may be a significant difference in the depth of rainfall and hence of runoff. This difference may be either in the storm that is studied or in the typical pattern of the basin, that may not be uniform in a watershed so orientated to the sea, and in storms influenced by such rugged ridges that parallel these many long tributaries. If


FIG. 11.—KARNAFULI WATERSHED

the typical or the specific distribution happened to be light in the narrow zone of the watershed and heavy in a wider portion, there could certainly be strong influences in the direction of double peaks.

The author also furnished some daily precipitation reports for July 28 to August 2, 1947, showing 3 in. to 32 in. in the downstream portion of the watershed, but actually no data at all for the remote areas of the watershed. The amounts are shown on Fig. 11. If there were little or no rainfall on the extreme limit of the watershed, and these large quantities in the half of the watershed nearest the gage, the writer has no doubt that the basic hydrograph for short duration would exhibit two peaks of the relative magnitude found by the author. But he would have to raise the question as to whether this non-uniform distribution could produce a multiple of a unit hydrograph as such a device is defined.

There is another possibility which makes this stream even more worth knowing about. It could have a double peak with the first peak being greater than

he second and for a uniformly distributed runoff, the basically defined distri- bution for a unitgraph, if the slope of the downstream waterway is much great- er than the upper one, or if it cuts through a gorge or deep bank much narrow- er than is upstream. Improbable as this might seem, the Karnafuli is a water- shed produced by some kind of uplifting and folding, and this action may have produced long flat ponding reaches above Barkal, or series of rapids and gorges below, or both. Until something more is known about the topography of this watershed, these possibilities must be kept in mind. These are the things that are determined by field reconnaissance and that often escape the desk-bound practitioner. One can hope that the future years can yield some records of the things which now must be surmised.

Quite obviously, the Karnafuli is not a simple stream of a homogeneous nature in a sedimentary bed, both because the author has described it otherwise, and because it could not exhibit the kind of stream pattern that it does if it were. As seen from the pattern of Fig. 11, there are long narrow valleys in the directions adverse to general stream direction, and an abundance of evidence of stream piracy and other forms of channels not in equilibrium with the simpler forces of stream building. If it were a simple sedimentary stream, it would not exhibit a unitgraph with a pronounced double peak. One may glimpse the factors which must be defined before streamflow chronology can be correlated with watershed characteristics and some of the problems of defining unitgraphs consistently, by comparing the hydrograph for such a simple watershed with that which is reported for this stream.

As presented in Fig. 12, the writer has derived a unit hydrograph, a storage-based instantaneous unit hydrograph (IUH), compatible with the furnished records of this stream, and the approximate shape of this watershed as defined by this map, Fig. 11, for a uniformly distributed unit quantity of simultaneously generated runoff. Engineers given the same watershed map, which is dimensionless, the same storage constants, and the same dimensional value of the total watershed area, could get the same answer, achieving the independence from the personal equation sought by the author in the present paper.

Comparing this uniform IUH function further with the author's derived unitgraph, the indicated peak is near the magnitude found by the author, and the essential duration is nearly that found by the author. Few currently active procedures concern themselves with more, and much useful work can be done with this much accuracy and agreement.

The agreement goes farther. Both falling legs have similar rates of recession. The author has placed considerable emphasis on the recession rate in making his derivation, and the users of the instantaneous unit hydrograph procedure recognize that one of the two storage constants basic to that method is a measure of this phenomenon. With so much agreement, it is almost axiomatic that the centers of gravity of the hydrographs and their lags behind rainfall are almost identical.

There still remains the questions of the exact hydrograph configuration and the sharp double peaks. The writer was genuinely disappointed that the basin shape, as he divided it with no real knowledge of the location of the watershed ridge lines, and many doubts about tributary connections, did not support the double peaks.

Indeed, the enlarged lower basin in comparison with the central zone near Barkal shows only as a shoulder on the rising hydrograph, not a peak. That which shows as a shoulder for a uniformly distributed short duration runoff would be a peak for some non-uniform rainfall distribution, perhaps, particularly with little or no runoff from the remote portions of the watershed. But

the rainfall distribution vaguely suggested by the few numerical values of rainfall shown also on Fig. 11, amounting to 32 in. somewhat above the mid-area although only 3.1 in. in the mid-zone, has a tendency in the opposite direction as shown by the non-uniform IUH function also shown on Fig. 12. The non-uniform IUH function is computed in much the same manner as the uniform

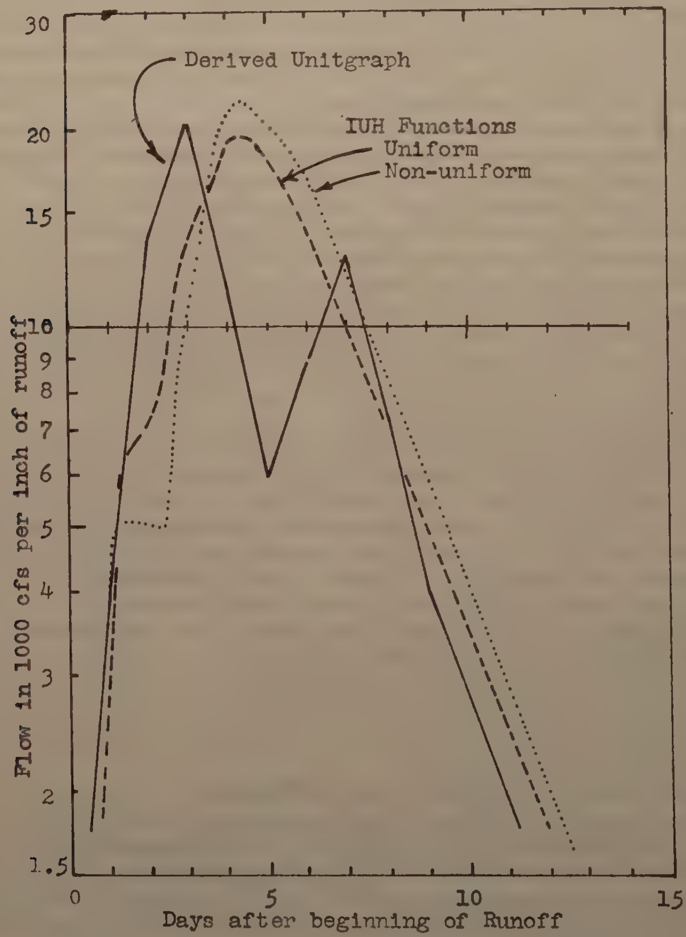


FIG. 12.—COMPARISON OF UNITHYDROGRAPH FUNCTIONS

IUH function, but with the modification of Victor H. Cochrane[3] to account fo varying depth of runoff.

The author's search for consistency in unitgraphs hinges on two significan procedures of derivation and the reduction of the result to a dimensionles

[3] "Storage and the Unit Hydrograph," by C. O. Clark, Transactions, ASCE, Vol. 11 1945, Discussion by Victor H. Cochrane, p. 1461.

asis. The procedure of derivation involving determination of volume moments f the hydrograph, and of the rainfall volume, and their difference as a meaure of the phase difference, or time lag, between runoff and stream flow is ound mathematics and a growingly popular procedure. It has a directly comarable relation to procedure in storage-based IUH functions.

The procedure of developing the unit hydrograph from a multiple - unit event f unknown non-uniformity of runoff distribution can be a necessary expedient, ut it is a mathematically indeterminate procedure in principle, capable of ndicating obvious errors like the negative flow values treated by the author, nd of producing a large number of sometimes conflicting sequences of numers which evade the consistency sought. Only the author's statement that the ouble peaks of his unitgraph "were present in all eight unitgraphs that were erived," and the assumption that this refers to unitgraphs from eight different flood events, leads the writer to hope for future evidence of the watershed auses of this indication. Some years ago the indeterminate aspect of this rocedure was discussed.[3] It hinges around the fact that in Eq. 3, when runff sequence is known, there are more equations for simultaneous solution than here are unknowns, and one set of unitgraph sequence values ($U_1$, $U_2$, $U_3$ . . . $U_n$) for each combination of equations can be found, all of which compare with ach other as differently as the forward and the reverse runs presented. The rst and the last series of the available equations make up the "forward run" nd the "reverse run" and are the series most commonly selected for this kind f analysis, but the intermediate series are available, and neither more nor ess accurate than the initial and terminal series. It appears probable that e solution values oscillate back and forth across the correct solution in some anner. The writer would be unable to estimate a measure of the amplitude f this oscillating deviation, and has found it to be large and occasionally embarrassing.

The author's addition to the procedure of the recession value adds another quation to those available, and another series of solutions to those considered bove. While this addition increases the number of solutions available, there s little doubt that it improves the quality of all solutions by substantially reucing the amplitude of deviation about the correct solution of the unitgraph.

If this process of simultaneous equation solution is extended to very short nits of time, fractions of the 24-hr unit illustrated, the number of equations or solution and the work of solution multiplies to a degree which precludes its erformance until rapid electronic computers became available. The instananeous unit hydrograph functions, IUH functions, with their infinitesimally hort unit periods and continuous curve of solution, were a natural, but a forced, volution from these difficulties.

The author recognizes lag, or a time-phase difference between runoff and e hydrograph, as a characteristic likely to define a hydrograph and susceptile to a successful correlation between watersheds and topographic measureents. Successful correlation with those watershed dimensions indicative of torage and discharge capacity has been obtained, and correlation with slope lso insofar as this item can define storage, discharge, or similarity of watereds. The measures of lag mentioned by the author refer in general to a ean-time displacement of the centers or centroids of hydrographs, rainfall, nd watersheds. Those contemplated in IUH functions are measures of the imilar displacement of the most remote elements of rainfall and watershed. Time of concentration" is the oldest presently used name representative of uch a concept, and the one most likely to lead to consistency over the widest

range of possible watershed dimensions. Presumably, for a rectangular watershed, the former group of displacement measures would be about one-half of the latter, and values between one third and two thirds of the latter might apply in the two kinds of triangular watersheds which define reasonable limits within which orthodox watershed shapes lie.

The author seems to have selected 108 - 12 = 96 hr as his measure of lag. The illustrated IUH function is for time of concentration of 144 hr. Hence the former is two thirds of the latter. The basin shape is such that the centroid of the watershed is beyond the half-length of the watershed. Greater consistency than this must await better means of defining and measuring the items and concepts under discussion.

The author defines the recession of the Karnafuli unitgraph by the dimensional time reciprocal, 0.69 per day. The writer defines the IUH recessional by the storage-discharge parameter 60 hr of 2.75 days, for which the recessional decrease is measured by the ratio of values 24 hr apart as 0.692. Apparently these are consistent, and are merely two ways of writing the same thing.

The author approaches consistency between watersheds via the dimensionless hydrograph. Merrill Bernard's distribution graph had this merit and it seems odd that it did not survive in place of the unit hydrograph. Perhaps it may be revived for its real merit. The IUH function approaches the same end via the dimensionless map or watershed shape diagram, commonly called distance-area curve. The shape diagram and the two storage parameters, days and 2.75 days for Karnafuli River above Rangamati, an area of about 380 sq miles, define the IUH function, either in the practically dimensionless units of percentage of total volume per hour, or in English units of cubic feet per second per watershed inch of volume. For this particular watershed the results have been shown to be similar in many respects, highly consistent by either, both, or any criterion of consistency.

The author demonstrates a range of application which also includes the much smaller Castle Creek having one thousandth the watershed. For IUH functions storage values of 8 min and 5 min have been found to define an instantaneous unitgraph for 1/4 acre near Temple, Texas, which is dry 99% of the time. Values of 5 days and 4.5 years similarly define the wholly theoretical unitgraph of the 300,000 sq miles involving the Great Lakes of the St. Lawrence River, where backwater effect from downstream points extends almost two thirds of the waterway length at low water, and 30% of the watershed is inundated at low water. The seeker of consistency may find it equally in these approaches and over wide ranges of possible use. The seeker of greater consistency must look also for more precise words, thoughts, and concepts, and first conceive and then measure the much smaller deviations in which these two approaches are not entirely identical and not completely consistent.

All streams are not alike, and some cannot be understood in the exact terms of a unitgraph, but that item is the best presently available tool for measuring a concept. Even where a unitgraph is not the rule or the exact nature of a particular stream, the unitgraph is the best hope of a yardstick about which and from which some other behavior can be defined in a consistent manner. An understanding of the factors presented by the author against the background of the reader's observed personal experience will lead to the consistency which has been attained only among civil engineers and within the Society. More records from this unique stream in Pakistan can help to close the few remaining gaps in a complete path to consistency in unitgraphs.

# REVISED COMPUTATION OF A VELOCITY HEAD WEIGHTED VALUE[a]

## Corrections to Discussion by Manual A. Benson

CORRECTIONS TO DISCUSSION BY MANUAL A. BENSON, A. M. ASCE.— the February Journal of the Hydraulics Division on page 128 the following orrections must be made: In the equation at the top of the page the term $A_2{}^2/A_1$) should be changed to $(A_2/A_1)^2$. In Table 1, Col. 3, change the heading "Conveyance, b" to "Conveyance, k."

In Table 1 the notation "Approximate Values of" that presently runs over olumns 5 through 8 should only encompass columns 5 through 7.

[a] September, 1959, by J. M. Lara and K. B. Schroeder.

# WAVE INDUCED MOTION OF BOTTOM SEDIMENT PARTICLES[a]

Discussion by Robert L. Miller

ROBERT L. MILLER.[1]—The sequential papers by Ippen and Eagleson[2] agleson, Dean and Peralta[3] and that under present discussion, have a direct d important bearing on a group of problems of interest to marine geologists, cluding the present writer. The interaction of shoaling waves and sediment-uid interface in the near shore region of the oceans presents a very complex oblem even under greatly simplified conditions. Specific questions in this tegory include: 1) equilibrium conditions for shorelines, in particular, beach-; 2) the mechanism of entrainment and subsequent deposition of sediment rticles; and 3) the resulting statistical patterns of average (or median) size d the variance (sorting) of the sediment particles.[4]

The first two papers referred to offer, to the marine geologist, the oppor-nity to test and interpret the null point hypothesis directly in nature. This as done by analysis of detailed field data collected specifically for this pur-se. The analysis was made on a number of field localities ranging[5,6,7] from e coast of California, to Lake Michigan, to the New England coast.

Expectation of the areal pattern for median size, and sorting (using the coef-ient of variation) was developed from the empirical relation given by Ippen d Eagleson[2] as,

$$\left(\frac{H}{d}\right)^2\left(\frac{L}{H}\right)\left(\frac{C}{W}\right) = 11.6 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

assuming an initial size distribution consistent with geological expectation. e final result was compared with field data subsequently smoothed by least uares. Consistently good agreement with expectation was found, provided

[a] October, 1959, by P. S. Eagleson and R. G. Dean.

[1] Assoc. Prof., Dept. of Geology, Univ. of Chicago, and Assoc, in Marine Geology, ods Hole Oceanographic Inst.

[2] "A Study of Sediment Sorting by Waves Shoaling on a Plane Beach," by A. T. Ippen l P. S. Eagleson, M. I. T. Hydrodynamics Lab. Tech. Report No. 18, 1955.

[3] "The Mechanics of the Mition of Discrete Spherical Bottom Sediment Particles due Shoaling Waves," by P. S. Eagleson, R. G. Dean and L. A. Peralta, Beach Erosion ard Tech. Memorandum No. 104, Washington, 1958.

[4] The average size, and the variance around the average of a sample of sediment ticles, constitute two very useful and widely used parameters in Geology.

[5] "A Model Relating Dynamics and Sediment Pattern in Equilibrium in the Region of aling Waves, Breaker Zone and Foreshore," by R. L. Miller and J. M. Zeigler, rnal of Geology, Vol. 66, No. 4, July, 1958.

[6] "A Study of the Relation Between Dynamics and Sediment Pattern," by R. L. Miller J. M. Zeigler, Eclogae Geologicae Helvitiae, Basel, Switzerland (in press).

[7] "Comparison of Theoretical Mass Transport Velocities with Observed Sediment e and Sorting Patterns," by R. L. Miller and J. M. Zeigler, First Internatl. Oceano-phic Congress Preprints, New York, N. Y., 1959.

certain simplifying assumptions were made and limiting conditions satisfied. The conclusion is that the relationship expressed in Eq. 1 appears to hold in nature under the restrictions referred to above.

In their present paper Eagleson and Dean present two rational equations for predicting median sediment diameter versus bottom slope, that are based, in part, on their experimental data, and in part on theoretical considerations including that of Longuet-Higgins.[8]

In view of the encouraging results achieved in applying the rational equation (Eq. 1) to natural conditions, it is planned to repeat the procedure with the new prediction equations of Eagleson and Dean.

However, there remains one important aspect that it is felt has been neglected by Eagleson and Dean. There is a definite pattern to the sorting or variation of the sediment in the near shore region, as well as in the median size distribution, in the natural cases which were studied.

Eagleson and Dean assume that $\frac{D}{k} = 1$ most nearly represents the bed particle geometry of equilibrium natural beaches, in which D is the sediment particle diameter, and k is the mean diameter of the roughness particle. This does not appear to be justified in nature. The ratio $\frac{D}{k}$ may be more realistically expressed as an average in a natural beach, but will also vary as a function of sorting. Since there is a definite gradient to the sorting pattern, an average $\frac{D}{k}$ will exhibit the greatest divergence from unity, in the seaward direction.

The movement of particles down-slope in a direction seaward of the various null points has been recorded under laboratory conditions. The existence of this phenomenon, to any appreciable degree in nature, is open to question. The following argument is given to support this contention.

Suppose we start with an initial uniform particle distribution. Then, as null points for the various sizes are subsequently established under wave motion, two possibilities are considered:

A. Particles smaller than a given null point move shoreward. Particles larger than a given null point move seaward.

B. Particles smaller than a given null point move shoreward. Particles larger than a given null point have no appreciable net movement.

In the first possibility, the implication is that at the null point itself the particle sizes remaining will all be close to that null point diameter. An evaluation of variation in dimensionless form, that is, standard deviation to mean deviation, should result in approximately equal values over a traverse from beach out to sea.

In the second possibility, the implication is that the ratio standard deviation to mean should show a definite gradient, from high values seaward to low values shoreward.

Observations in nature and those of others such as Inman[9] indicate a definite gradient from high values of the dimensionless sorting coefficient seaward to low values shoreward. Thus B appears to be correct, and seaward movement of particles down slope from null points appears to be negligible.

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8 "Mass Transport in Water Waves," by Longnet-Higgens, Philosophical Transactions, London, Ser. A, No. 903, Vol. 245, March, 1953, pp. 535-581.

9 "Areal and Seasonal Variations in Beach and Nearshore Sediments at La Jolla, California," by D. L. Inman, Beach Erosion Board Tech. Memorandum No. 34, 1953.

If the implications of the gradient in sorting are considered, then, on the verage D/k varies from the value of 1, with the greatest divergence in a sea-ard direction. In such a case the statistical chance that a particle move down-lope under the influence of gravity is minimized in nature, due to the low like-hood of particle removal from "pockets" at the interface, and the high like-hood of immediate retrapping of particles in "pockets" when they are moved 1 a downslope direction. As D/k approaches unity the similarity between the atural condition and the experiments performed at the Massachusetts Institute f Technology (M.I.T.) will become stronger.

In summary, the two new prediction equations given by Eagleson and Dean ppear to be considerably generalized and refined over the earlier form given 1 Ippen and Eagleson. Of particular importance for field investigation is the resent inclusion of beach slope as a variable, and the implicit inclusion of ass transport velocities.

The writer's collected field data will be reanalyzed in terms of these new rediction equations with the expectation of considerable improvement in our esults. It is felt that Eagleson and Dean have made a distinct contribution ward solution of the complex problem of sediment movements versus wave otion in the near shore region.

# HYDRAULIC CHARACTERISTICS OF GATE SLOTS[a]

Discussions by E. Roy Tinney, J. M. Robertson and H. W. Bennett

E. ROY TINNEY,[1] A. M. ASCE.—The analysis of flow in the vicinity of a ate slot has considerable practical significance even though the slot itself is minor detail in a large structure. Excessive vibration of the gate leaf and avitation damage to the downstream conduit walls can become serious opertional hazards.

The problem is essentially one of determining a slot shape that will acommodate a wheel, roller, or guide without affecting the flow. Although this night seem to be a straightforward problem, in reality it becomes somewhat omplex. The fundamental problems involved are: (1) the spreading of jets nd, (2) flow separation at discontinuities in the boundaries. The offset disance depends on the rate of spread of the jet into the slot and the shape of the ownstream corner must be designed so that the impinging high velocity jet oes not tend to separate from the downstream wall.

One experimental approach to this problem is to study the flow patterns in ne neighborhood of the slot with particular attention to the spreading of the jet nd separation of the flow. The author however, has chosen the alternate aproach of analyzing pressure patterns for several of the most practical designs ith a systematic variation of the dimensions and configurations. The sucess of this latter approach has been well demonstrated. It is evident from ne data presented that a gate slot with a small offset and a very gradual tranition from the downstream corner to the line of the downstream wall is avitation-free even at extremely high heads. The type of transition does not ppear to be critical as long as it is gradual since a 24 to 1 convergence with 12-in. radius arc at the downstream end appears to give results comparable ith those for a large radius, circular arc convergence (Fig. 14). Presumaly, a 1 to 5 ellipse would be equally satisfactory since it gives an acceptable olution even when it projects into the flow (Fig. 16). These findings are in ccord with designs for other cavitation-free bodies subject to impingement y high velocities, for example, baffles on spillway aprons, nose-bodies for orpedoes, struts, etc.

The writer questions the interpretation placed on the data on incipient caviation. In particular, the writer disagrees with the statement that "the presure on the surface just downstream of the downstream slot corner need not be f vapor pressure magnitude for cavitation to occur at the slot," and with the dea that a "scale effect" exists. Two other interpretations are suggested that ccount for the apparent anomalies. First, the average pressure as indicated y a piezometer-manometer tube apparatus may be considerably higher than

[a] October, 1959, by J. W. Ball.

[1] Head, The R. L. Albrook Hydr. Lab., Div. of Industrial Research, Washington State Jniv.

vapor pressure and yet the pressure may fluctuate so strongly that it will be at vapor pressure for a significant percentage of the time. Indeed, it has been shown[2] that model data may indicate average pressures as high as -14.5 ft of water and yet serious cavitation damage occurs on the prototype. The use of pressure transducers and suitable recording instruments to measure pressure fluctuations show that at this same point the instantaneous pressure was less than -30 ft of water for 26% of the time.

A second explanation for the apparent discrepancy between relatively high pressures as measured on the model and the occurrence of cavitation is the extreme difficulty in finding the minimum pressure when the pressure gradients are large. On Fig. 11, for example, it is not at all clear that the minimum value of $(h_x - h_o)/h_v$ for Slot A is -0.16 as indicated. It may be as low as -0.30. If this latter value is used with the data on Fig. 16 for the square corner ($h_v$ = 146, $h_o$ = 14), the minimum value of $h_x$ is -30 ft of water which agrees well with the plotted data. It does not appear, therefore, that a "scale effect" exists as the author suggests.

The alternate method proposed by the author of using a cavitation index as determined by special model tests of course presupposes the absence of "scale effect." This technique does not require detailed pressure measurement but it does require careful visual observation. A compilation of such indices for the common types of gate slots would greatly simplify the design of cavitation-free gate slots. In the meantime, the application of the excellent set of guide lines presented in this paper is most helpful and the author is to be commended for the diligent effort that he has made over a long period of time to gather this wealth of material.

J. M. ROBERTSON,[3] M. ASCE and H. W. BENNETT.[4]—The author presents a thorough discussion of the hydraulic problems and their possible solutions resulting from the necessity of having gate slots in large conduits. His test results and observations represent a valuable extension of our knowledge of the flow past slots and generally agree with those from other studies. This discussion is based primarily on the writers' experience with a slot problem in a large hydraulic structure where the experimental results supplement the author's. The necessity for information on the boundary-layer thickness in the flow approaching the slot is emphasized.

The paper discusses two basic hydraulic gate-slot problems—"turbulence" within the slopes and cavitation in or just downstream of the slots. Although these may result in entirely different damage effects, they are not unrelated. Both are intimitely associated with the flow conditions in and near the slot. The vortex flow pattern (that the author terms turbulence) in the slot is a function of the slot proportions—primarily the width to depth ratio—as suggested by the author's observation that a great amount of circulation occurs in the slots and that this varies with slot width. In view of other observations on the flow pattern in slots, the comment that the circulation seems greater with wide slots than with narrower ones must be considered to be of limited applicability. Certainly the circulation or vorticity is small for very wide slots, since i

2 "Hydraulic Model Studies of Noxon Rapids Stilling Basin," by Howard D. Copp and E. Roy Tinney. Unpublished report by The R. L. Albrook Hydr. Lab., Div. of Industrial Research, Washington State Univ., February, 1960.

3 Prof. of Theoretical and Applied Mech., Univ. of Illinois, Urbana, Ill.

4 Tech. Supt., Riverside Paper Corp., Appleton, Wis.

uch a case the flow appears to expand into the slot with definite vortices only ear the ends.

The nature of vortex occurrences in slots is such that one must be careful n interpreting the results of experiments conducted on a few width-depth atios. The vortex structure and some of the resulting effects may vary in a early discontinuous fashion with the slot proportions. For certain slot proortions (W/D ratio about one, one-half, or three) rather stable patterns apear likely. With proportions not greatly different, the vortex pattern is not efinite and considerable flow disturbance is possible. An unstable pattern apears likely for a depth to width ratio of about two. For rather deep slots the orticity is small and the flow disturbance negligible. This is the rational xplanation for the common rule that a piezometer hole for pressure measure-

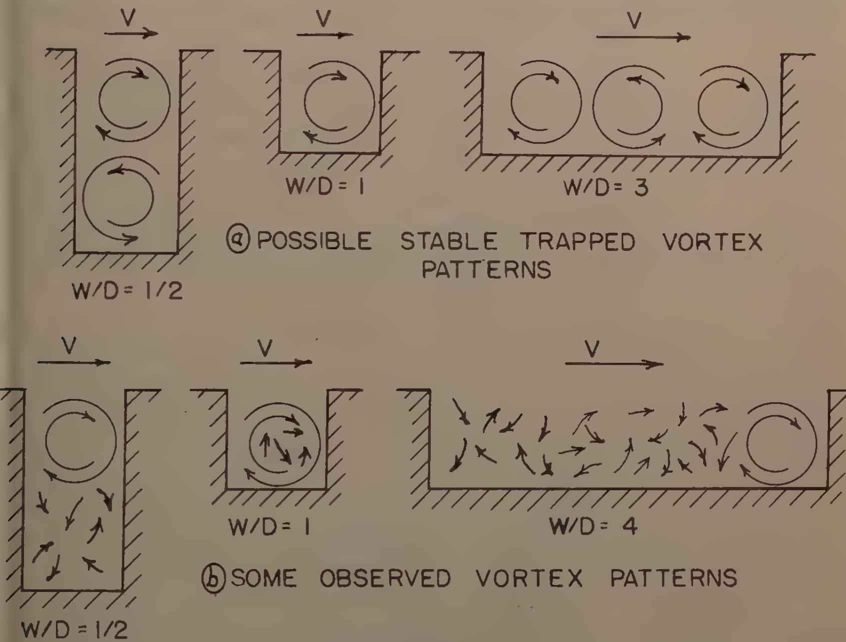


FIG. 1.—INTERNAL FLOW PATTERNS IN SLOTS OF VARIOUS PROPORTIONS

ent should be at least two diameters deep before it changes in size. Fig. 1(a) dicates possible stable vortex patterns for this and shallower (that is, wider) ots. Very few recorded observations of the vortex patterns in slots are availle, although numerous authors indicate the stable trapped vortex found with W/D ratio of about unity. Thom and Appelt have recently computed[5] the minar-viscous flow pattern at low slot Reynolds numbers indicating the maner in which the dividing stream line enters into the openings. In connection th consideration of the flow past rough surfaces, when the protuberances re-

[5] "The Pressure in a Two-Dimensional Static Pressure Hole at Low Reylonds Numrs," by A. Thom and G. J. Appelt, British Aero. Res. Council, ARC18798-FM2473, bruary 20, 1957.

sult in large W/D values, a few experimentors have indicated[6,7] a pattern of one, or possibly two, vortices behind a protruding obstacle with a following region of disturbed flow into which the outer flow intrudes. This is in general agreement with the writers' observations[8] as indicated in Fig. 1(b). Folsom reported on motion pictures of the unsteady pattern showing successive formation and sweeping out of vortices. In his studies, the maximum energy loss occurred when the width was about twice the depth. One can rationalize that a vortex pair pattern, which might be expected for W/D of about two in the scheme of stable trapped vortices indicated in Fig. 1(a), is not possible due to the manner in which the flow must occur at the upstream and downstream corners. At least one other flow situation, of which the writers are cognizant, has exhibited an almost discontinuous behavior (as evidenced by a loss coefficient) at a certain W/D ratio. The careful studies[9] by Roshko show some discontinuities in slot flow at W/D ratios in the vicinity of 2.

Returning to the observations of the pressure distributions immediately downstream of slots, as shown in the author's Fig. 6, one might question whether the curves indicated might not actually contain discontinuities. Since the maximum disturbance (from energy loss measurements) appears at a width to depth ratio of about two, some discontinuities in the pressure curves might be anticipated near this ratio value. That such are not in evidence, either in the curves shown by the author or presented later in this discussion, may only be due to an incompleteness in the experimental programs from which the data was obtained. Certainly any future experimental studies of this problem should verify the possibility of discontinuous behavior.

As the author indicates, the effect of offsetting the corners of the slots may be either beneficial or adverse depending upon the relative offset direction. Evidence of offset effects obtained at the Iowa Institute of Hydraulic Research has shown[10,11] that as the upstream corner is raised the pressure dip is quickly reduced while raising the downstream corner increases the dip. This is in qualitative agreement with the author's Fig. 6. Some interesting "field" or prototype experience on slot offset effects was obtained with the Garfield Thomas Water Tunnel at the Pennsylvania State University. In this research facility, water at speeds up to some 80 fps flows through a 4-ft cylindrical section (about 14 ft long) in which hydroballistic shapes are tested for their cavitation and other characteristics. Access to the test section is by way of a 2.5 ft by 10 ft removable section or cover at the top of the 4-ft conduit. The necessary clearance slots at the upstream and downstream edges of this cover, although only about 1/16 in. wide, were of great concern in the design of the tunnel due to their being possible sources of cavitation. Since no offsetting was expected,

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6 "Frictional Resistance of Artificially Roughened Pipes," discussion by R. G. Folsom, Transactions, ASCE, Vol. 101, 1936, p. 707.

7 "Relation of Statistical Theory of Turbulence to Hydraulics," discussion by P. Nemenyi, Transactions, ASCE, Vol. 105, 1940, p. 1582.

8 "Flow Past a Slot in a Surface," by H. W. Bennett, The Pennsylvania State Univ. Master's thesis, 1949.

9 "Some Measurements of Flow in a Rectangular Cutout," by A. Roshko, NACA TN 3488, August, 1955.

10 "Use of the Low Velocity Air Tunnel in Hydraulic Research," by H. Rouse, Proceedings, 3rd Hydr. Conference, Studies in Engrg., Bulletin 31, State Univ. of Iowa, 1947, p. 132.

11 "Cavitation at Sluice Gate Slots," by A. Fetouh, Master's thesis, State Univ. of Iowa, 1947.

design studies indicated that any problem could be easily minimized via a rounding of the downstream corners.

Studies of the cavitation characteristics of the completed tunnel indicated the occurrence of premature cavitation. This was finally traced to an offset in the seating of the cover, resulting from an improper gasket construction. The offset was measured to be about 0.050 in., which is a rather small amount in the 4-ft diameter. It was an appreciable fraction of the boundary-layer displacement thickness, that was about 0.11 in. at the front of the cover where the offset was out of the flow and 0.24 in. at the end where the offset was into the flow. After reworking of the gasket to eliminate this offset, the test section cavitation index or number was found[12] to be reduced from 1.1 to a value of about 0.3. It is probably coincidental that this final value of the cavitation index is so close to the author's value of 0.28 for the rectangular slot. One very important aspect of the water tunnel slot cavitation occurrence was that it was only evidenced, and presumably only occurred, at speeds above 45 fps. The implications of such a discontinuous speed effect are serious when one considers extrapolation of low-speed cavitation tests to structures at other scales.[13]

The evaluation of cavitation potentialities of structures from experiments involving wall-pressure measurements are indicated by the author in his discussion of the results found with the standard slide gate. The minimum wall-pressure coefficient was found not to be the same as the cavitation index, presumably found by visual observation of cavitation inception. Such a discrepancy may be due to several factors, particularly those involving the effects of physical properties and occurrences, such as are often lumped under the cognomen "scale effects." One factor, which is not really a scale effect, is that cavitation probably did not occur along the wall where the pressures were measured, but in the separated flow region away from the wall. This same situation may not occur for well rounded contours which inhibit separation. Consideration of the studies of other hydraulic structures susceptible to cavitation suggests that the best solution is not always found by reducing the magnitude of the wall pressure dip, since this may lead to cavity collapse on the surface rather than off the surface (that is, in the water). Cavitation collapse in the separated flow off the surface may result in less cavitation erosion, even though more severe cavitation occurs. Thus we have the problem of correlating our conclusions based on wall-pressure measurements with cavitation inception and erosion occurrences. Until these several problems are sorted out, we must temper our use of wall pressure measurements with qualitative estimates of the nature of the flow behavior and how it may affect cavitation.

Most experimenters in hydraulic structures, including the author, appear to neglect one factor which can have a significant effect on their results and the proper interpretation thereof. This is the nature of the boundary layer flow near the contour they are studying. In the present case, prototype conduit structures do not have uniform velocity profiles nor is the flow always fully developed. In the laboratory studies no consideration appears to have been given to simulating, or even evaluating the effect of, the nature (primarily the relative boundary-layer thickness) of the approaching flow. As will be shown,

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12 "Performance Characteristics of a 48 inch Water Tunnel," by J. M. Robertson and . Ross, Engineering, Vol. 184, July 19, 1957, p. 76.

13 "Similitude et Cavitation–on Several Laws of Cavitation Scaling," by J. M. Robertson, J. H. McGinley and J. W. Holl, La Houille Blanche, September, 1957, p. 540.

the boundary-layer thickness at a slot can have a significant effect on the flow and pressure distribution. Relatively thick boundary layers tend to smooth over sharp discontinuities in contour. Thin layers do not and hence result in large effects approaching those predictable from potential flow analyses.

Information on the affects of boundary-layer thickness on the wall pressure distributions near wall slots was obtained[8] some time ago as part of the design studies for the large high-speed water tunnel already referred to. The slot of concern in the prototype was that around the access cover in the test section. The experiments were performed on slot models much larger than the prototype in order that the flow could be studied in detail. A small single-pass air tunnel was used for the study with velocities outside the boundary layer of about 90 fps. The slot widths were 1 in. to 4 in. leading to testing at slot Reynolds numbers of about the same values as expected in the prototype. The tunnel

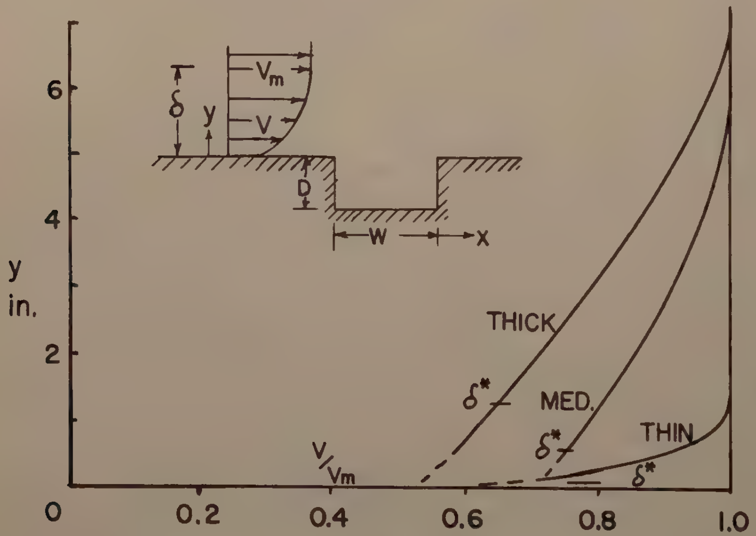


FIG. 2.—BOUNDARY LAYER VELOCITY PROFILES

cross section at the test location was 3 in. by 10 in. and the slot extended across the 3-in. wide floor. Since one factor of major concern was the boundary-layer thickness, three different boundary layers were developed—one natural and two artificially thickened. The natural layer, here termed the thin layer, was about 1.2 in. thick with a displacement thickness $\delta$ of about 0.14 in. The other two boundary layers had velocity profiles that agreed reasonably well with those for natural profiles; the ratio of displacement to momentum thickness, a conventional indicator of the nature of the profile, was 1.14 and 1.29, not far from the usual value of about 1.3. The medium boundary layer had a thickness of about 5.5 in. and a displacement thickness of 0.64 in., while for the thick layer these thicknesses were about 7 in. and 1.28 in. The three velocity profiles are shown in Fig. 2. These three boundary layers, in conjunction with the use of slot widths of 1 in. to 4 in., permitted a study of the problem over a wide range in relative boundary layer thickness. Besides the flow pattern observations already noted, the wall pressure distribution in and near the slot was also observed.

Wall pressure distribution results similar to those shown by the author were ound for various combinations of slot depth, width, and boundary-layer thick- ess. Tests were made with square-edged and rounded downstream corners. n all cases the upstream corner was sharp and without offset. The effect of lot width, boundary-layer thickness, and corner rounding were clearly evi- enced. Over a range in width to depth ratios of 1/4 to 4 the slot depth appeared o have little effect, as shown in Fig. 3. The pressure coefficient $C_p$ used here s similar to the coefficient plotted by the author except that the measured ressures have been corrected for the hydraulic gradient in the conduit (that s, in the test tunnel) and the dynamic pressure by which the pressures are di- ided is based on the velocity outside the boundary layer. The proper sign

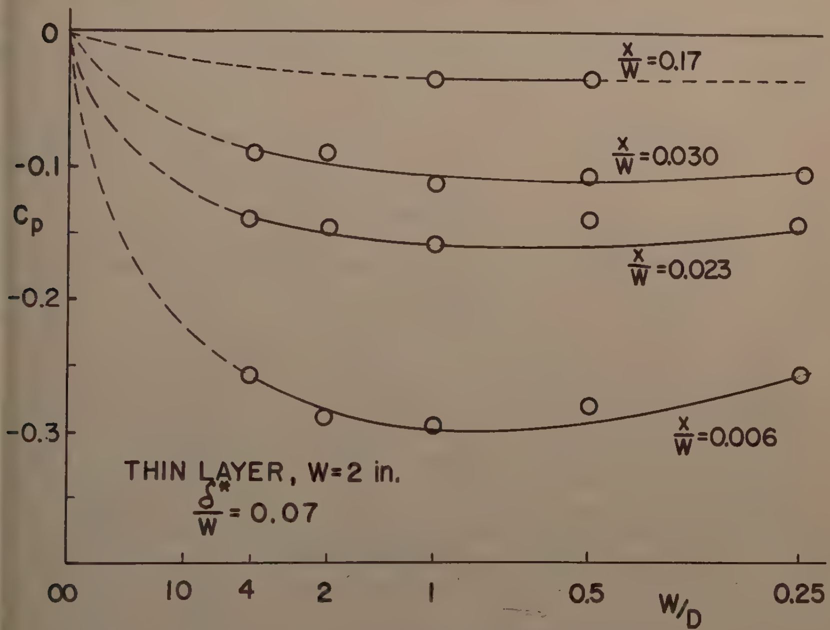


FIG. 3.—EFFECT OF RELATIVE SLOT WIDTH ON WALL PRESSURE FOLLOWING SLOT

onvention (in contrast to the author's inverted one) has been used with this oefficient. Thus, pressures below the hydraulic grade line appear as nega- ve coefficients. Although the smooth curves drawn through the points in ig. 3 appear to contravene the earlier discussion of slot proportions' effects nd discontinuous occurrences, they do appear reasonable. Actually, a more ontinuous variation in W/D should have been studied. The lack of significant lot depth effect, as noted, differs from the author's observations as shown in ig. 6F. As will be discussed below, pressures' measurements downstream f sharp corners probably only indicate the minimum if they are within a dis- ance of about 0.01 slot widths of the corner. This implies that the author's iezometer number 4 only indicated the minimum pressure when W was 6 in. nd larger.

Slot width and boundary-layer thickness had a marked effect on the pressure distributions. This is clearly shown for one of the rounded corners in Fig. 4. For the wider of the two slots the pressure patterns have a maximum located about 45° around the curve while for the narrower slot the maximum occurs further around the curve (in the flow direction), that is, at about 70°. The ef-

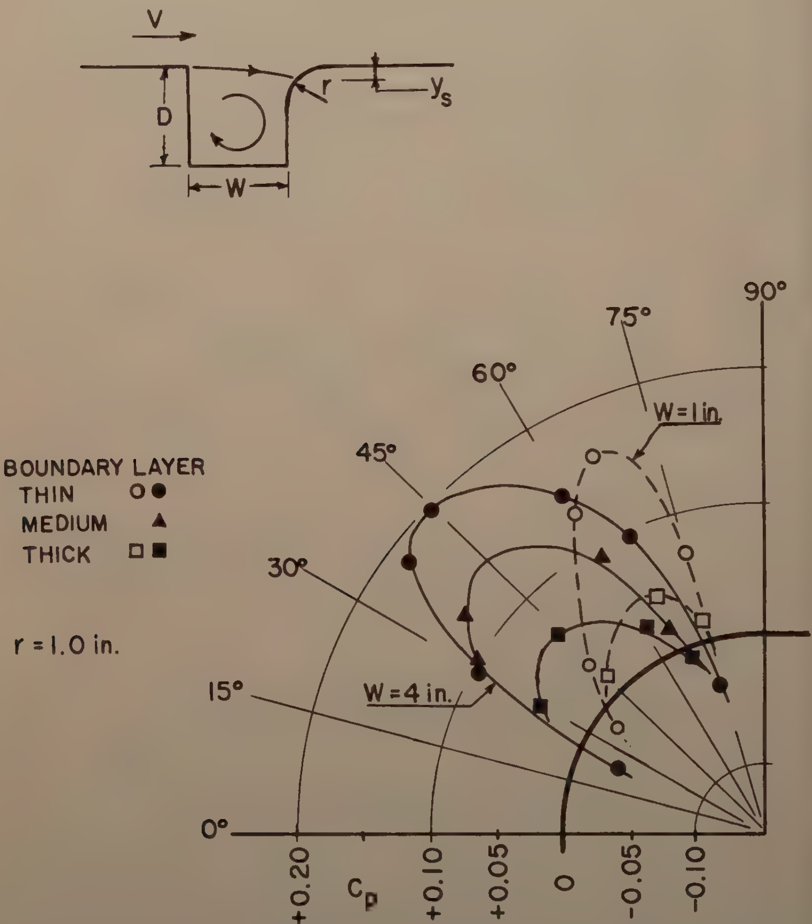


FIG. 4.—PRESSURE DISTRIBUTION AROUND RADIUSED CORNER AT END OF SLOT

fect of boundary-layer thickness is seen to be that of modulating the amplitud of the pressures. The higher pressures occur with the thinner boundary layers

Proper determination of the minimum wall-pressure coefficient require that the first piezometer following the corner lies in the region of minimur pressure. For the sharp cornered slots the extent of this region must be de fined by the separation of the flow from the corner. This in turn is depender

on the location of the point where the streamline separating the outer flow from hat trapped in the slot intersects the downstream wall of the slot. This point s a stagnation point whose location is indicated by the location of the maximum pressure in plots such as that of Fig. 4. Although much more difficult of observation, a similar positive pressure peak appeared for square cornered slots. The distance $-y_s$ of this stagnation point below the upper surface of the slot is proportional to the width of the slot, as noted in connection with Fig. 4. It appeared, in fact, to occur at a location indicated by $-y_s/W = 0.006$ for both sharp and rounded corners. From study of various other flows with separation it appears the region of separation should extend along the wall in the flow direction a distance about 1.5 times $y_s$. This suggests that the separation (and hence minimum pressure) zone extends to an x/W of about 0.01. For the tests reported in this discussion the minimum pressures were thus only indicated by he first piezometer (at x = 0.013 in.) when W was 2 in. and 4 in. In general, great care must be exercised in interpreting pressure measurements made at a fixed distance from the corner of the slot as the width of the slot is changed. t thus appears that some of the author's results may be misleading due to the above consideration.

Taking data from the first piezometer following the 2 in. and 4 in. wide lots as representing the minimum pressure, it is possible to indicate the effect of relative boundary-layer thickness on the pressure dip for the sharp orner. This is shown in the upper plot of Fig. 5 together with some of the esults for the rounded corners. The importance of the boundary-layer thickness in determining the minimum pressure following a slot, particularly a harp cornered one, is clearly apparent. The thinner the boundary layer relaive to the slot width, the larger the magnitude of the pressure dip.

The suitability of providing some downstream corner radii in order to reuce the magnitude of the pressure dip is indicated by the lower plot of Fig. 5. This shows clearly the importance of a small rounding of the corner in allevi-ting the pressure dip. It should be noted that only a simple radiusing of the orners was studied in the tests reported here. The author shows clearly the ffect of curving this corner with transitions more gentle than the simple circuar arc. A substantial decrease in the magnitude of the pressure dip is obtained, ut a secondary dip occurs because the curve did not joint the wall of the conuit properly. The water tunnel design studies previously referred to included study of transition curves[14] in which it was indicated that contours must be tilized which have continuous first and second derivatives. Thus a cubic tranition curve is desirable. The first derivative should be continuous to avoid eparation while the second should be to avoid pressure discontinuities as preicted by potential flow theory. Electrical analogy studies of conduit inlets[15] y Rouse and Hassan also indicate the desirability of cubic transition curves. f course just how sophisticated a curve is necessary depends upon the relave boundary-layer thickness. The cubic may only be needed for very thin oundary layers such as will be encountered near conduit entrances.

If this discussion does nothing else, it is hoped that it shows clearly the importance of the effect of boundary-layer thickness on the slot problem. Thus many of the author's results (those of Fig. 20, for example) are of uncertain bsolute validity. The boundary-layer thickness for a given test situation is

14 "Water Tunnel Transition-Section Flow Studies," by D. Ross, Ordnance Research ab. Report NOrd 7958-285, The Pennsylvania State Univ., 1954.

15 "Cavitation-Free Inlets and Contractions," by H. Rouse and M. M. Hassan, Mechanal Engineering, March, 1953, p. 213.

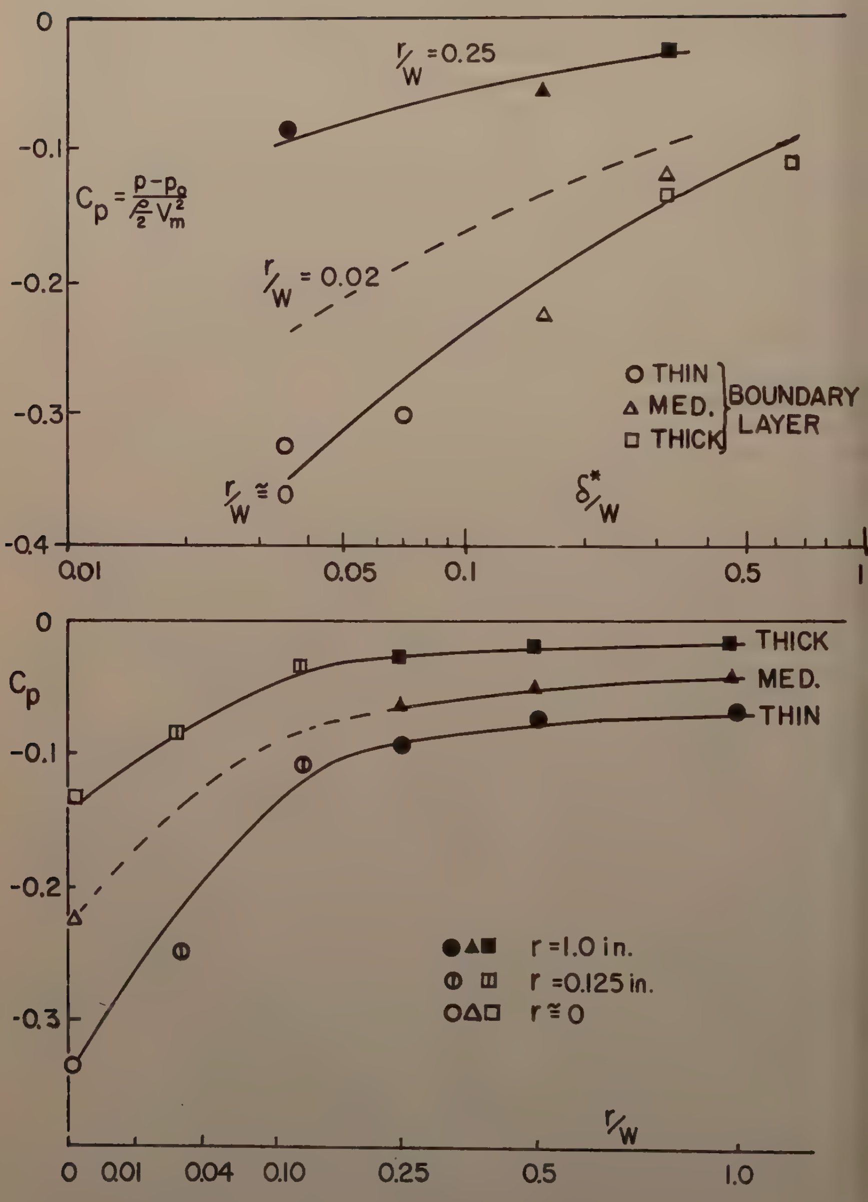


FIG. 5.—EFFECT OF BOUNDARY LAYER THICKNESS AND CORNER RADIUS ON MINIMUM WALL PRESSURE

as much a part of specifying the test conditions as is the width of the slot. It is hoped that the author will be able to indicate some estimates of the relevant boundary-layer thicknesses. Thus, the test data leading to the curves of Fig. 20 was presumably obtained at one station in a conduit or tunnel of known geometry. If this had a well-rounded entry and known length of straight section preceeding the offsets, it should be possible to estimate the approximate boundary-layer displacement thickness. It would seem from the information presented in this discussion that any such estimate would make Fig. 20 of considerable absolute value whereas now it is of little such value. Actually, of course, the effect of variations in boundary-layer thickness on this particular occurrence should be studied.

# CAVITATION DAMAGE OF ROUGHENED CONCRETE SURFACES[a]

Discussion by A. Rylands Thomas

A. RYLANDS THOMAS,[1] F. ASCE.—The relationship between shear velocity nd pressure head (Fig. 8) provides a useful indication of the conditions under hich cavitation is likely to occur. The margin of safety against cavitation rovided by static pressure is clearly illustrated.

There is, of course, no clear-cut "threshold" of cavitation. Turbulence is random phenomenon and therefore the onset of cavitation in a turbulent stream a discontinuous process which could be defined by frequency of occurrence. may be that the build-up of cavitation to considerable proportions is rapid nd that for practical purposes a diagram such as Fig. 8 is all that is required.

The tacit assumption in the application of results obtained in a conduit 3 in. eep to larger conduits (see Fig. 10) is that the instantaneous velocity at the oundary at the onset of cavitation is dependent only on the boundary roughness nd on the shear velocity at that point, and not on flow parameters relating to e whole cross section, such as for Reynolds Number of mean flow. It could e pointed out that the conditions slightly away from the boundary are not identical because the rate of decrease of shear velocity with distance from the oundary is greater in a small conduit than in a large one. Nevertheless, the ssumption appears to be justified by observational data of the intensities of elocities of turbulence near the boundaries of rough and smooth conduits, the oot-mean-squares of which bear a constant ratio to the shear velocity for y given boundary surface.

The curves shown in Figs. 8 and 10 relate only to two specimens of eroded ncrete surface tested by the author. Useful as these are, the designer of gh-velocity tunnels and open conduits requires information relating to the set of cavitation before erosion of the surface has occurred. In particular, requires information on smooth concrete surfaces with protuberances or eps due to joints in formwork and slight waviness due to lack of alignment of rms. It is hoped that the author's work will be extended to cover these practical cases.

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[a] November, 1959, by Donald Colgate.
[1] Cons., London, England.

# MOUNTAIN CHANNEL TREATMENT IN LOS ANGELES COUNTY[a]

Discussion by George N. Newhall and Frank M. Henry

GEORGE N. NEWHALL,[1] M. ASCE, and FRANK M. HENRY.[2]—Mr. Ferrell's aper gives a very clear background and summary of the objectives, benefits, osts, and methods of the valuable channel-treatment program underway in .os Angeles River Watershed, California.

This writer, having viewed the Brand Canyon, Cooks Canyon, and Monroe anyon works, and part of the other works and reference reports, offers the ›llowing comments and questions with a view of advancing a few small facets f the great and largely inexact science of watershed management engineering.

Increased deep percolation of ground water could be, as Mr. Ferrell point-d out, a very important resulting benefit of such channel treatment projects areas of fractured or porous rock formations. It is conceivable that, if the ›cations where maximum or significant deep percolation occurs can be found, e amount of deep percolation could be further beneficially increased, at rela-vely small cost by either a) increasing the height of dam at, or just down-ream from, that location, or b) drilling tunnels or wells from the channel ıst upstream from a dam, preferably before the reach of channel fills with ebris. Such tunnels or wells could be horizontal, vertical, or inclined to fol-w the strike and dip of rock strata and fault zones, and could extend in one more directions from the channel, depending on the geology, hydrology, and :onomics of the project. Have such features been considered, investigated, planned in connection with this program on mountain channels in Los Angeles ›unty? And have estimates been made of probable increased amounts of deep ›rcolation?

The significant amount of "inflow water" within many water supply tunnels, ıch as Tecolote, Mission (Gibraltar) and Doulton (Juncal) Tunnels in the Santa arbara area is evidence that valuable amounts of usable water can and do me out of deep percolation below some mountain watersheds. Amounts from mgd to 10 mgd flow fairly steadily throughout the year, year after year, with ›re in seasons following heavy precipitation seasons, and less in drought pe-›ds. So it seems logical that this "deep percolation groundwater" could be tificially increased through tunnels or wells or barriers, just as it can be tracted from tunnels or wells or springs.

For such "recharge" purposes, floodwaters would need to be de-silted or latively free of silt, as has been discovered by experience of Los Angeles ›unty Flood Control District and various irrigation districts with recharge

[a] November, 1959, by W. R. Ferrell.
[1] Civ. Engr., U. S. Forest Service, Santa Barbara, Calif.
[2] Civ. Engr., U. S. Forest Service, San Francisco, Calif.

well operation. Such de-silting or debris load reduction of flood waters is ap-parently accomplished to some extent automatically in channel stabilizatio projects such as Brand Canyon and Cooks Canyon as a result of decreased ve-locities of flood flows. This clearer water may be an inherent benefit of suc projects, due partly to resulting natural revegetation of channels, and it lead to the conclusion that any proposed "recharge tunnels" or well intakes shoul be several feet lower than dam spillway level, so as to get clear "recharg water."

This benefit is unpredictable without a thorough study of the geological for-mations and a prediction as to quantity of underground streamflows and thei probable pattern. This can well be a sizable benefit and should be given mor consideration during planning. The feasibility of installing tunnels, and so forth for collecting and distributing underground waters would be considered in th cost studies. A primary consideration would be whether or not the additiona waters would be seasonal (intermittent) or continuous. Perhaps the studies o Monroe Canyon (and others) may serve as guidelines.

Mr. Ferrell points out that it is anticipated, after channel stabilization, "tha any debris that does enter the reservoir will be fine material which can b more readily sluiced." Whether this material is sluiced through a debris dar or excavated from a debris basin, it would seem that this finer material, hav-ing come from mountain-watershed topsoil and duff, would often contain desir-able plant nutrients, and might be economically salvaged and utilized by nur-series, ranchers, gardeners, or park departments as topsoil if it can be ef-fectively advertised, stockpiled, and distributed. This could in some case result in income greater than the costs of advertising, stockpiling, and distri-bution, which "net income" or profit could be applied against channel stabili-zation project costs.

This may well be a local condition, that is, on watersheds with ideal cove humus, and so forth, that will furnish a high percentage of desirable silt a related to the total eroded material. The cost of excavating and transportin selected basin deposits may be too expensive inasmuch as removal costs ma approach $3.00 per cubic yard. This is not a planning item but is one to b considered at the time of basin sluicing or debris removal.

Mr. Ferrell quotes one policy item adopted about 1956 by his District a "the spillway height should not be less than 10 ft above the stream bed no greater than 17 ft." It is not logical that some dams should be higher than 1 ft? These could be the dams where groundwater conservation benefits woul be great enough to influence the design and economics. Or perhaps they woul be dams located at the better damsites, where favorable topography might indi-cate that a dam higher than 17 ft would be more economical. It is recognize that other factors may also control the maximum desirable height of these de-bris barriers, such as safety, standard crib member dimensions, or other de-sign criteria resulting from model studies and stream profile studies. Howeve further study of the benefits of increasing deep percolation for water conse vation may show that dams or barriers higher than 17 ft may be optimum certain locations.

# ELECTRONIC COMPUTERS USED FOR HYDROLOGIC PROBLEMS[a]

Discussion by Alexandre Preissmann

ALEXANDRE PREISSMANN.[1]—The authors are to be congratulated for having shown how important the use of electronic calculating equipment can be for a very wide variety of hydrologic problems. Calculations for which desk computers appeared to be hopelessly inadequate require comparatively little time and expenditure if solved by means of electronic equipment.

It is felt that the authors could have gone even further, for the use of electronic calculators can quite appreciably change the ways and methods of approaching hydrologic problems in rivers, as well as setting new problems. It is hoped that the following examples may help to present this idea more explicitly.

Before predicting the overall summer flow from meteorological and physiographic data, multiple correlations have to be established. If only desk computers are available, a very small amount of data, likely to determine the overall summer flow (such as an estimate of the snow cover), are introduced in the regressions. However, if electronic equipment is available, one may be tempted to introduce large amounts of data in the forecast equations, and then to pick out from all the possible regressions the one corresponding to the highest total correlation factor. Although such a method would be acceptable if the available records covered an extremely large number of years, this requirement is unfortunately never met in real life. The available records only extend over very few years, and computed correlation factors are affected by random errors that are quite appreciable.

The highest computed correlation factor out of a hundred does correspond to the best forecast equation ever likely to correspond to the highest true total correlation coefficient.

Unless one chooses a poor prediction method—or, in extreme cases, one that is actually absurd—one must be in possession of criteria enabling a reasonable choice to be made. For instance, one could ask oneself whether the formulas obtained for a given basin can reasonably be applied to other similar basins (that implies an increased use of calculating equipment) in which case the considered formulas would become more reliable. An alternative method would be to discard any formulas which cannot be interpreted physically. Whatever the case under consideration, it is not true to say that the use of high-speed calculating machines supersedes the hydrologist's experience. On the contrary, it gives it its full meaning.

Flood problems are approached by analyzing the records and attempting to use the observations to find empirical relationships between the lag of time

[a] November, 1959, by Francis E. Swain and Herbert S. Riesbol.

[1] Engr. at S.O.G.R.E.A.H. (Societe Grenobloise d'Etudes et d'Applications Hydrauliques), Grenoble, France.

between two points, or the spreading out of the flood, and certain data, such as the season or the magnitude of the flood. Where electronic equipment is available, this may not necessarily be the most rational approach, for such equipment can be used to construct a mathematical "model" of unsteady flow of the river, based on the principles of conservation of mass and momentum. By adjusting certain data (coefficients of frictional resistance), one can arrive at a rational understanding of the observed effects. By running such a "model"—which would be quite out of the question with ordinary desk computers—the effect of man-made changes to the course of the river (such as storage in upstream reservoirs, dykes, cutoffs, and so forth) upon the flood regime can be estimated to within a good degree of approximation.

It would appear, therefore, that the available electronic calculating machines now may be expected not only to modify appreciably the details of computation procedure, but also—and this is much more important—to give rise to problems of a new type requiring new methods to solve them.

In particular, the use of electronic equipment for systematic studies of a large number of basins may well make it possible, in the long run, to extend empirical rules originally established for individual basins to any basins of a certain type, and thus to gain a clearer insight into the runoff phenomenon. In this connection, it is felt that users of electronic calculating equipment for hydrologic problems might benefit by discussing their difficulties and results at periodic intervals.

# HYDRAULIC DOWNPULL FORCES ON HIGH HEAD GATES[a]

Discussions by Robert G. Cox and Ellis B. Pickett, and W. P. Simmons, Jr.

ROBERT G. COX,[1] M. ASCE and ELLIS B. PICKETT,[2] A. M. ASCE.—The author has presented a valuable and interesting paper on the vertical hydraulic forces on face and cylinder gates. There is a definite need for more information on this subject, especially if the data can be expressed in dimensionless parameters suitable for design purposes.

This discussion is limited to control gates which are normally located some distance downstream from the sluice or tunnel entrance. The control gate recess is either an open gate well or a bonnet-type recess. Fig. 1 shows the features of these two types of recesses.

Factors considered in this discussion are: 1) The effects of gate clearances and venting on the hydraulic force on the top of the gate; 2) The effects of gate bottom geometry and venting on the hydraulic force on the bottom of the gate; 3) The development of dimensionless parameters for design; and 4) The effects of gate bottom geometry and venting on the stability of the gate.

The equation for the hoist load resulting from the hydraulic and gravity forces acting on the gate shown on Fig. 2 is

$$P = W + A\gamma\left(d_f - u_f\right) \qquad (1)$$

in which P denotes the hydraulic and gravity forces in tons, W is the dry weight of gate in tons, A signifies cross-sectional area of gate in square feet, $d_f$ denotes average unit-of-area downthrust on top of gate in feet of water, $u_f$ equals average unit-of-area upthrust on bottom of gate in feet of water, and $\gamma$ is the specific weight of water, 0.0312 tons per $ft^3$. The friction load is an additional force that should be considered in actual design. The friction load is dependent on the pressure drop from the upstream to the downstream side of the gate, the face area of the gate, and the rolling or sliding coefficient of the materials in contact.

The weight of the gate is not included in the data presented here but a review of buoyancy and "wet weight" of the gate is of interest. Fig. 2 simplifies definition of the hydraulic forces involved. For the purpose of discussing buoyancy, a gate may be assumed to be a rectangular parallelepiped with the vertical axis coincident with the direction of gravity. If the body is completely in-

[a] November, 1959, by Donald Cogate.

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[3] "Outlet Works and Spillway for Garrison Dam, Missouri River, North Dakota, Corps of Engrs., U. S. Army, Waterways Experiment Sta., Tech. Memorandum 2-431, March, 1956.

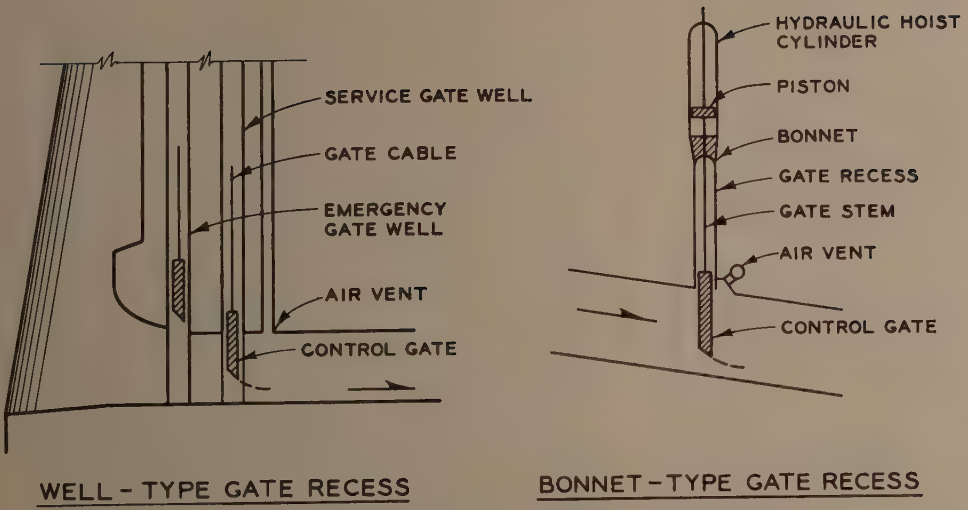



FIG. 1.—CONTROL GATES IN TUNNELS AND SLUICES

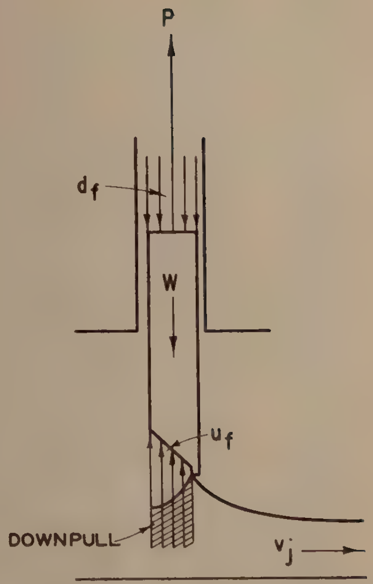



FIG. 2.—VERTICAL HYDRAULIC FORCES ACTING ON A CONTROL GATE

closed, the buoyant force in still water is equal to the difference between the total pressure on top (downthrust) and the total pressure on the bottom (upthrust). For such an inclosed body, water pressure on the upstream face has no vertical component of pressure.

Some engineers use the expression, the "wet weight" of a gate. This is simply the dry weight in air minus the buoyant force. If the body is cellular or lacks an upstream skin plate, the wet weight differs from that of the completely inclosed body shown in Fig. 2. The gate shown is further considered to have no horizontal projections such as gate seals.

*Tests and Measurements*.—The U. S. Army Corps of Engineer Waterways Experiment Station, has made a number of tests, both model and prototype, to evaluate the vertical hydraulic forces acting on control gates. Measurements of gate oscillations resulting from fluctuations in these forces have also been made. All test results presented in this discussion are for fixed gate openings and all dimensions and forces are prototype values or dimensionless.

The total hydraulic force acting on a gate in motion is slightly greater or less than that acting on a fixed gate depending upon whether the gate is being raised or lowered. A moving gate is accompanied by a lag in changes of the water depth or pressure on top of the gate and the discharge under the gate. However, the rate of travel of a gate is normally less than two ft per minute and the effects of gate motion on the hydraulic load are believed to be small.

The pressure on top of the gate, or downthrust, can be determined by measurement of the water-surface elevation in the gate well or by measurement of the pressure in the gate bonnet. The pressure, or upthrust, on the bottom of the gate can be measured by means of piezometer taps or electrical transducers located on the gate bottom. Electrical transducers also permit evaluation of the pressure fluctuations which contribute to gate oscillations. Gate oscillations can also be measured by accelerometers, strain gages and dynamometers installed in the gate system.

The average hydraulic load can be measured in models by spring scales. Dynamometers and strain gages are suitable to both model and prototype testing and permit evaluation of fluctuations in the hydraulic load as well as average values. Measurements with all these instruments normally include the weight of the gate and the effects of friction damping. Model gates can be designed so that the friction load is negligible. In the prototype this force usually is computed. However, it can be indirectly evaluated if the hoist load, downthrust, upthrust and weight of the gate are known. In models, the weight of the gate can be compensated for in calibration of the measuring equipment but would generally be computed in prototype tests. The prototype hoist load can be computed from the cylinder pressures in hydraulically operated gates.

*Downthrust*.—The unit pressure on top of a gate, or downthrust, is dependent on the geometry and size of the upstream and downstream clearances between the gate and the gate recess and the relative pressure differences across the gaps. The flow across the top of the gate has some hydrodynamic effect, but this is considered negligible. A series of tests were made[3] on the Garrison Dam model to determine the effect of gate clearance on the downthrust. In these tests the downstream clearance area remained constant at 6.3 sq ft while the upstream clearance area was reduced from 15.2 sq ft to 8, 6, and 4 sq ft. Fig. 3 shows the geometry and dimensions of the clearances tested. Fig. 4 shows the decrease in downthrust with each decrease in upstream clearance. The curves on this figure reflect the effect of changes in gap geometry as well as gap areas.

Air vents are provided downstream from control gates to relieve low pressures when back pressure from full conduit flow does not exist. If an air vent is not provided, the reduced pressure behind the gate may cause a lowering of the water surface in the gate well, a reduction in the pressure on the bottom of the gate, and a decrease in the stability of the gate.

The effect of an air vent on the pressure differential across the gaps is indicated by the curves on Fig. 5. The curves are based[4] on the Ft. Randall Dam model tests in which the downthrust was measured with the air vent downstream of the gate open and then closed. Pressures on the conduit side of the downstream gap were essentially atmospheric with the vent open. The pressure differential across this gap was greatly increased by closing the air vent. The water-surface elevation in the gate well lowered until the flows through the upstream and downstream gaps were again equalized. An appreciable decrease in the downthrust resulted.

| D | AREA |
|---|---|
| 0.583' | 4 $FT^2$ |
| 0.479' | 6 $FT^2$ |
| 0.375' | 8 $FT^2$ |
| 0.0 | 15.2 $FT^2$ |

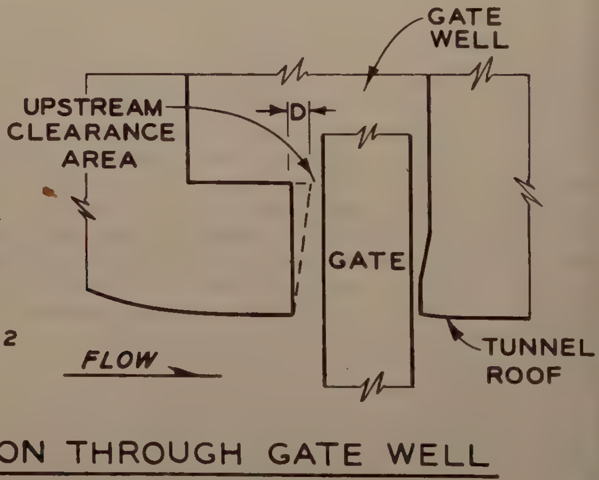



FIG. 3.—GEOMETRY OF CLEARANCES TESTED

The energy or friction losses of the flow through the gaps and, consequently, the downthrust is dependent upon the gap surface roughness and flow Reynolds numbers as well as gap areas and geometry. Model results should be transferable directly to prototype values if the model upstream and downstream gaps have the same proportion relative to each other as those of the prototype. However, the model and prototype flow quantities through the gaps do not necessarily have the correct scale ratio. If the designer has reliable data on gap discharge coefficients, the gate well water-surface elevation and, consequently, the downthrust could be computed for any gap geometry and pressure differential. Gap areas and shapes vary with each structure designed and frequently with each gate opening. Therefore, a series of laboratory tests to determine discharge loss coefficients for a number of gap sizes and shapes would be help-

[4] "Spillway and Outlet Works, Fort Randall Dam, Missouri River, South Dakota," Corps of Engrs., U.S. Army, Waterways Experiment Sta., Tech. Report No. 2-528, October, 1959.

ful to the designer. The necessary tests could be made on large-scale, two-dimensional gap models.

The gate well can be sucked completely dry of water with certain combinations of upstream and downstream gap areas between the gate and the roof of the conduit. If then, the upthrust exceeds the weight of the gate, the entire body of the gate will be thrust vertically upward. Such a phenomenon occurred at one Corps of Engineers project. Therefore, the gate recess should be designed so that the downthrust combined with the weight of the gate will exceed the upthrust on the bottom of the gate.

Another important design consideration is the high velocity jet flowing through the downstream gap and the accompanying local low pressures which could result in cavitation damage. The first few feet of the recess at the down-

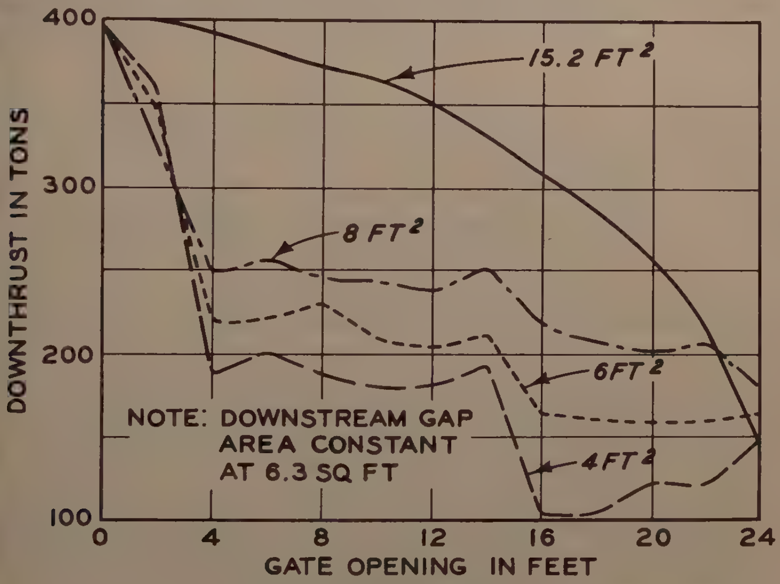



FIG. 4.—EFFECT OF UPSTREAM GAP AREA ON DOWNTHRUST

stream gap is sometimes steel-lined if excessively low local pressures are anticipated.

*Upthrust.*—The hydrodynamic effect of water flowing past the bottom of the gate is substantial. A reduction of pressure on the bottom from the theoretical static head is generally called "downpull" (Fig. 2). Downpull may be viewed either as a reduction in "upthrust" or a reduction in "buoyancy" and is dependent upon the geometry of the gate bottom.

The Waterways Experiment Station made a series of tests on a number of gate bottom designs as a part of the preliminary design studies for Garrison and Ft. Randall Dams. Complete descriptions of these tests are given[3,4] elsewhere. Seven of the gate designs tested are shown on Fig. 6. Special care was taken to reduce the gate friction load to a minimum by the use of steel roller bearings that contacted the gate slots in all directions and by the omission of gate seals. The total vertical hydraulic load was measured independently by

means of a spring scale and by strain gages. The submerged weight of the gate was compensated for during equipment calibration prior to beginning each test. The downthrust was computed from the observed water-surface elevations in the gate well. The upthrust was computed by substracting the downthrust from the total hydraulic load. Fig. 6 also shows the effects of gate bottom geometry on upthrust with the air vent open. Negative upthrust indicates less than atmospheric pressures on the bottom of the gate. For type 1 gate this pressure would amount to approximately -28 ft of water in the prototype and probably cavitation would occur on the gate bottom. Five of the designs tested involved extension of the downstream gate lip from 0 to 3 ft. Each increase in gate lip extension was accompanied by an increase in total upthrust. Type 13 is the standard design adopted by the Corps of Engineers. It is structurally more desirable than most of the other types and results in favorable upthrust. Type 14 is hydraulically adequate but the parabolic gate bottom curve presents fabrication difficulties. This design was installed at Denison Dam prior to design

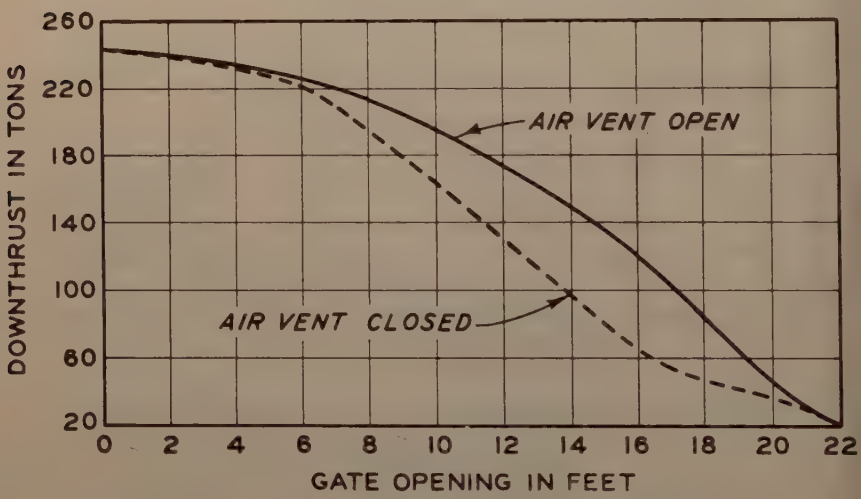



FIG. 5.—EFFECT OF DOWNSTREAM VENTING ON DOWNTHRUST

of type 13. The curves shown on Fig. 6 supplement those presented on the author's Figs. 2 and 3.

Fig. 7 presents upthrust curves for four of the model gate designs (see Fig. 6) with the downstream vent closed. Only type 13 had positive upthrust at all gate openings. It should be noted that upthrust values lower than about -40 tons for these model gates indicate prototype pressures lower than absolute zero. Therefore, the actual minimum upthrust on prototypes of these gates would be about -40 tons.

No discussion of upthrust would be complete without mentioning determination of gate bottom pressures by the flow net method. The technique for drawing flow nets have been discussed in numerous publications. A typical flow net for evaluation of pressures on the bottom of a gate is given[5] by Rouse

[5] "Engineering Hydraulics," by H. Rouse, John Wiley and Sons, Inc., New York and Chapman and Hall, Ltd., London, 1950, p. 538.

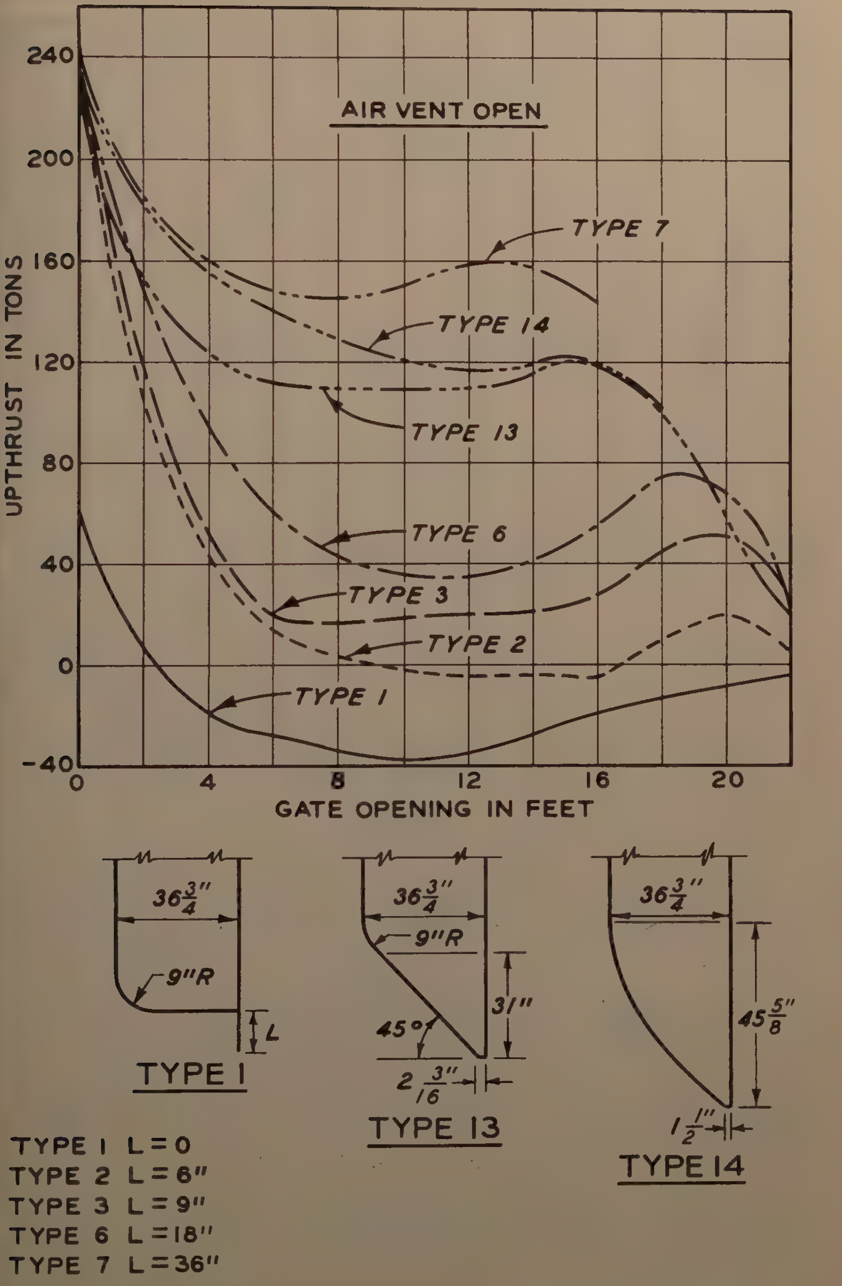



FIG. 6.—EFFECT OF GATE BOTTOM GEOMETRY ON UPTHRUST, AIR VENT OPEN

*Dimensionless Presentation.*—The data so far presented are dimensiona[l] and are not easily applicable to design problems. The Waterways Experimen[t] Station has prepared dimensionless design charts[6] on downthrust and upthrus[t] for use in the design offices of the Corps of Engineers. Simplifications of thes[e] charts are included as Figs. 8 and 9.

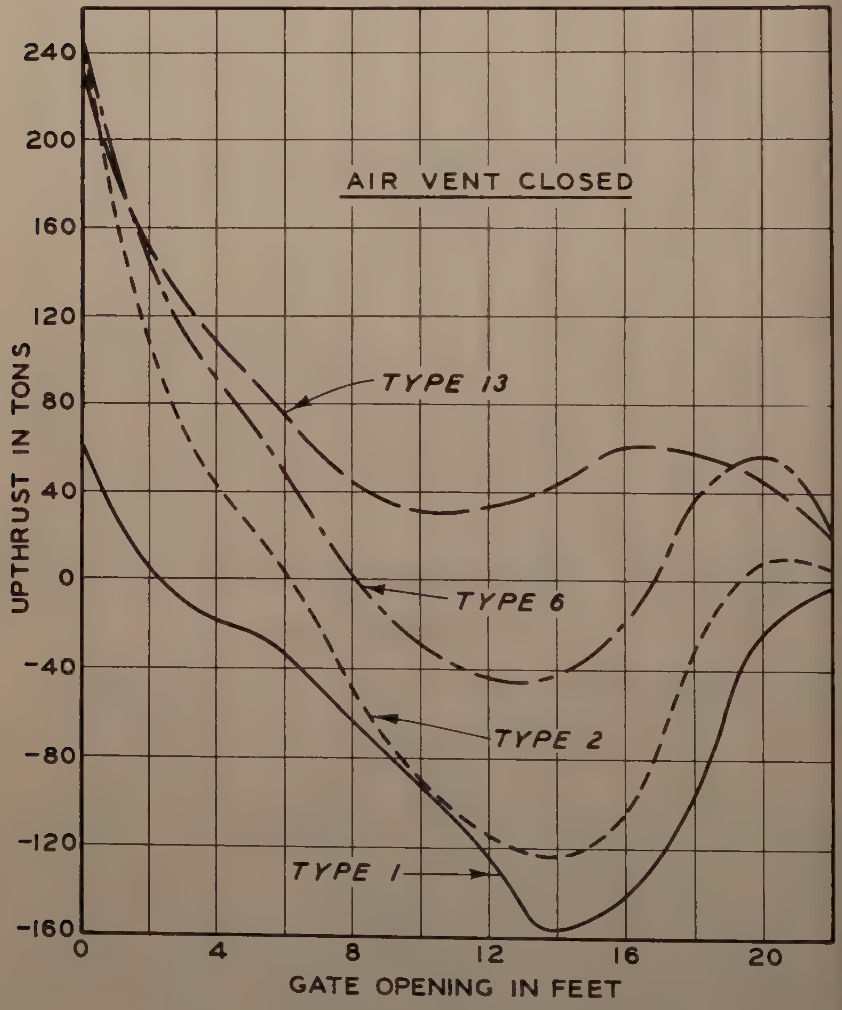



FIG. 7.—EFFECT OF GATE BOTTOM GEOMETRY ON UPTHRUST, AIR VENT CLOSED

6 "Hydraulic Design Criteria," Corps of Engrs., U. S. Army, Waterways Experime[nt] Sta.

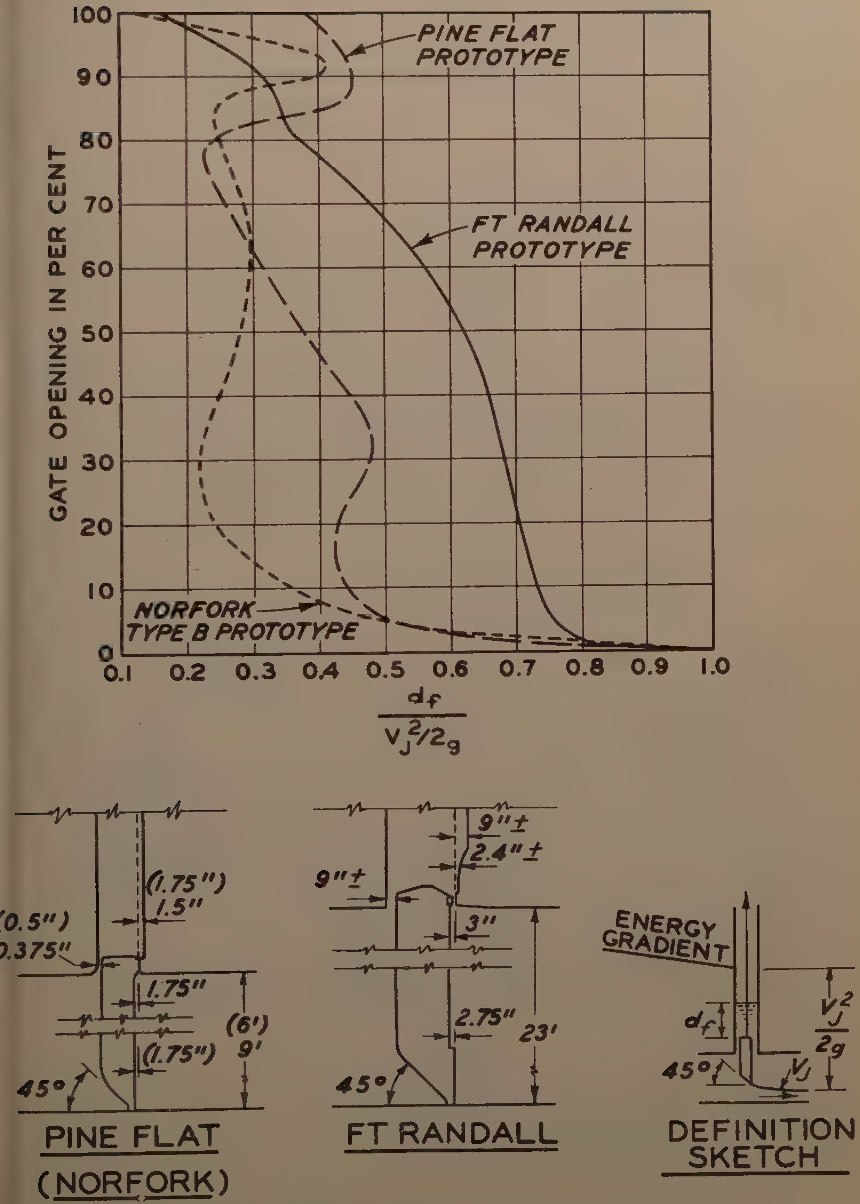
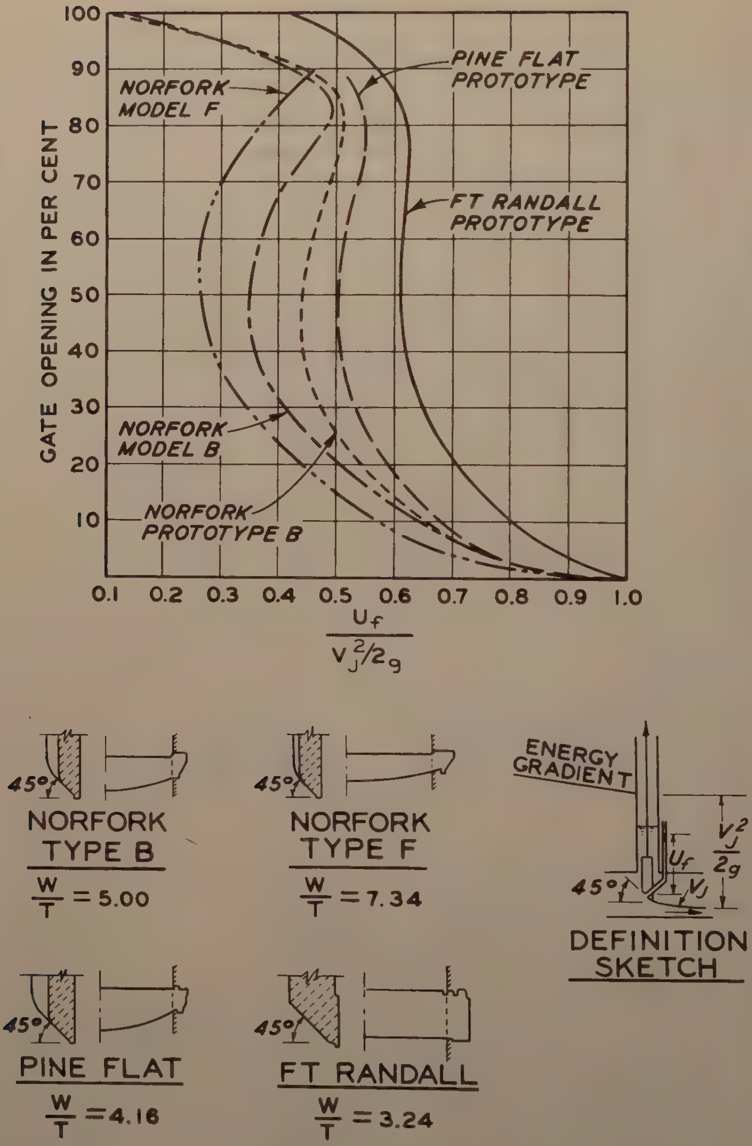



FIG. 8.—DIMENSIONLESS PLOT OF EFFECT OF RECESS GEOMETRY ON DOWNTHRUST




NOTE: T = AVERAGE THICKNESS OF GATE - FT

W = WIDTH OF CONDUIT - FT

FIG. 9.—DIMENSIONLESS PLOT OF EFFECT OF GATE BOTTOM GEOMETRY ON UPTHRUST

Fig. 8 shows dimensionless pressures on top of the gate. The curves are basedon field measurements[7,8,9] of gate-well water-surface elevations at Ft. Randall Dam, in South Dakota, and bonnet pressures at the Pine Flat, dam in California, and Norfork dam in Arkansas. The observed pressures ($d_f$) in feet of water were divided by the velocity head of the jet at the vena contracta ($V_j^2/2g$) to correlate data for different reservoir elevations. This velocity head was used because it excludes all energy losses upstream from the gate. Details of the clearances between the gate and gate recesses are also shown

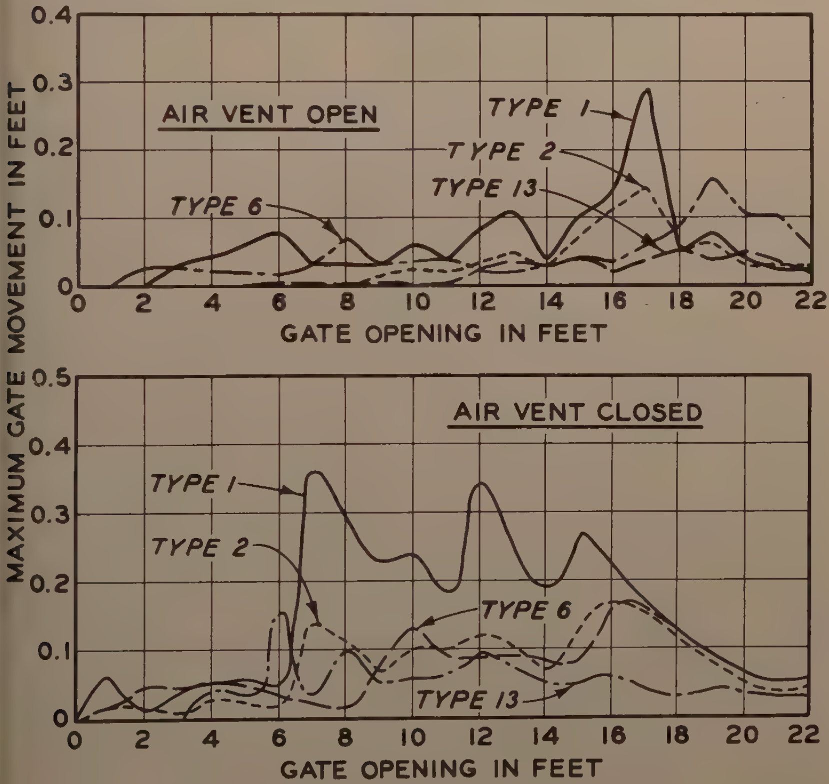



FIG. 10.—EFFECTS OF GATE BOTTOM GEOMETRY AND VENTING ON GATE OSCILLATIONS

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7 "Vibration and Pressure-Cell Tests, Flood-Control Intake Gates, Fort Randall Dam, Missouri River, South Dakota," Corps of Engrs., U. S. Army, Waterways Experiment Sta., Tech. Report No. 2-435, June, 1956.

8 "Vibration, Pressure and Air Demand Tests in Flood Control Sluice, Pine Flat Dam, Kings River, California," Corps of Engrs., U. S. Army, Waterways Experiment Sta., Miscellaneous Paper No. 2-75, February, 1954.

9 "Slide Gate Tests, Norfork Dam, North Fork River, Arkansas," Corps of Engrs., U. S. Army, Waterways Experiment Sta., Tech. Memorandum No. 2-389, July, 1954.

on this figure. The area to be used in computing the downthrust should includ the area of the gate within the gate slots, the area between the conduit wall and the area of the gate top seal. The large downthrust values for the Ft. Ran dall gate result from the fact that the upstream clearance is considerably great er than the downstream clearance. The smaller ratios of the gap clearance for the Pine Flat and Norfork gates resulted in an appreciable reduction i downthrust.

Fig. 9 shows similar dimensionless curves for upthrust on the sloping bot tom of four 45-degree gate-bottom designs. The Ft. Randall gate has a down stream skin plate on the gate proper, an upstream skin plate on the slopin gate bottom and downstream seals. The Pine Flat and Norfork gates have up stream skin plates and downstream seals.

The upthrust forces were computed from observed pressure data on th sloping gate bottom. Pressure contours were drawn, integrated, and divide by the projected gate area between the conduit walls to determine the upthrus ($u_f$) per unit area of cross section. The velocity head of the jet at the ven contracta was used to correlate data for different reservoir elevations. Th curves indicate that the ratio of conduit-width to average gate-thickness is factor in the magnitude of upthrust per unit area. The average gate thicknes includes the gate bottom seal.

*Stability*.—Vertical oscillations of the gate caused by pressure fluctuatio on the gate bottom are not usually considered in hoist equipment design. Thes oscillations alternately decrease and increase the hoist-load. The friction load should normally damp out minor gate oscillations. However, it is desir able to design gate bottom geometry for minimum vertical movement of th gate. Fig. 10 shows the maximum observed gate movement for type 1, 2, 6 an 13 gates with the air vent open and closed. The curves indicate the relativ stability that will be obtained with the 45° sloping gate bottom (type 13).

*Conclusions*.—The writers are in agreement with the conclusions of the au thor. Vertical hydraulic forces on gates based on model tests are needed fo economical design of hoist capacity. Confirmation of model test results b prototype measurements is always desirable. Considerable information i available on gate bottom pressures, but relatively little is known about gat clearance criteria and the resulting downthrust. Measurement of downthrus in prototype structures is relatively simple and it is hoped that such measure ments will be made whenever possible. For design purposes, it is necessar to compute the height of the water-surface in the gate well or the pressure hea in the bonnet. It is believed that discharge coefficients could be determine from a model investigation of gap areas and geometry, and that large-scal tests should minimize any scale effects. Measurements of the flow through th gaps together with pressure differentials would permit evaluation of the re quired discharge or loss coefficients. Gaps for existing prototype structure should be included in any general model investigation. The results of such series of tests would provide valuable data for designers.

W. P. SIMMONS, Jr.,[10] M. ASCE.—The author clearly points out a numbe of difficulties encountered with the direct weighing method of determining dowr pull in tests of high head gates that are fully submerged. The usual use of pressure tank for these high heads in place of a high and cumbersome head bo

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[10] Hydr. Engrg. Research, Bur. of Reclamation, Denver, Colo.

introduces friction at the point where the gate lifting stem emerges from the tank. In this respect, the high head tests differ from the simpler ones often conducted for spillway gates,[11] and so forth. In the latter case, downpull measurements by direct reading of scales or similar equipment are possible without any such frictional interference on the stem itself.

Strain gages similar to the ones mentioned in the prototype studies for the Shasta dam may be placed on the lifting stems of models within the pressure tank. They, thus, offer a partial solution to the problem in that they measure the load inside the tank and eliminate any stem friction from the load measurements. They do not eliminate another major problem posed by the sliding or rolling friction between the leaf itself and its supporting tracks or guides.

Several excellent methods for reducing or eliminating this leaf to frame friction have been developed.[11,12] However, all these methods seem to have the effect of releasing the seals from the seating surface, thereby permitting leakage. Leakage past the top seal can appreciably alter the measurement of heads acting on the top of the leaf in high head gates because it will change the depth of water within the bonnet or shaft and, hence, over the leaf top. Leakage past either the top or the side seals of the gate can influence the flow pattern in filled tunnels downstream from the leaf and at the leaf bottom. Unfortunately, the effects of leakage are relatively indeterminant factors, but they are, nevertheless, real and must be considered.

Downpull determinations by the pressure-area method also have limitations. Adequate coverage of representative areas of the leaf bottom by many piezometers is mandatory but not always easily obtained. Frequently, a sizeable percentage of the gate leaf is situated within the gate slots. This is particularly true of fixed-wheel and roller-train gates that are built up from standard structural rolled beams and plates and require very wide slots. The pressures acting on the portions of the leaf within the slots are markedly different from those in the main flow passage. Furthermore, the pressure gradient from a point just inside the slot to a point deeply inside may be steep and of unpredictable pattern. Only extensive piezometric coverage will suffice to produce reasonably accurate downpull results. To further complicate the situation, appurtenances like box enclosures around the roller trains or flow cutoff plates below the fixed wheels must be considered and included. In the direct measurement method, the effects of these variables do not require special treatment, other than being certain that the gate is truly represented in the model, because their effects are automatically included in the load measurement.

Difficulty may also be encountered in deciding what the pressure acting downward on the top of the gate should be and where it will act. The problem is simple in the case of a fully enclosed, downstream seal leaf because the pressures in the bonnet can be measured and referred to the elevation of the gate top. But in the case where the leaf is built up from structural members and has an open upstream face, it may be impossible to say where the downward force may be considered to act and, hence, what the effective force will be. In addition there may be appreciable downward drag force due to the action of high velocity water on the exposed horizontal beams. Thus, the net value

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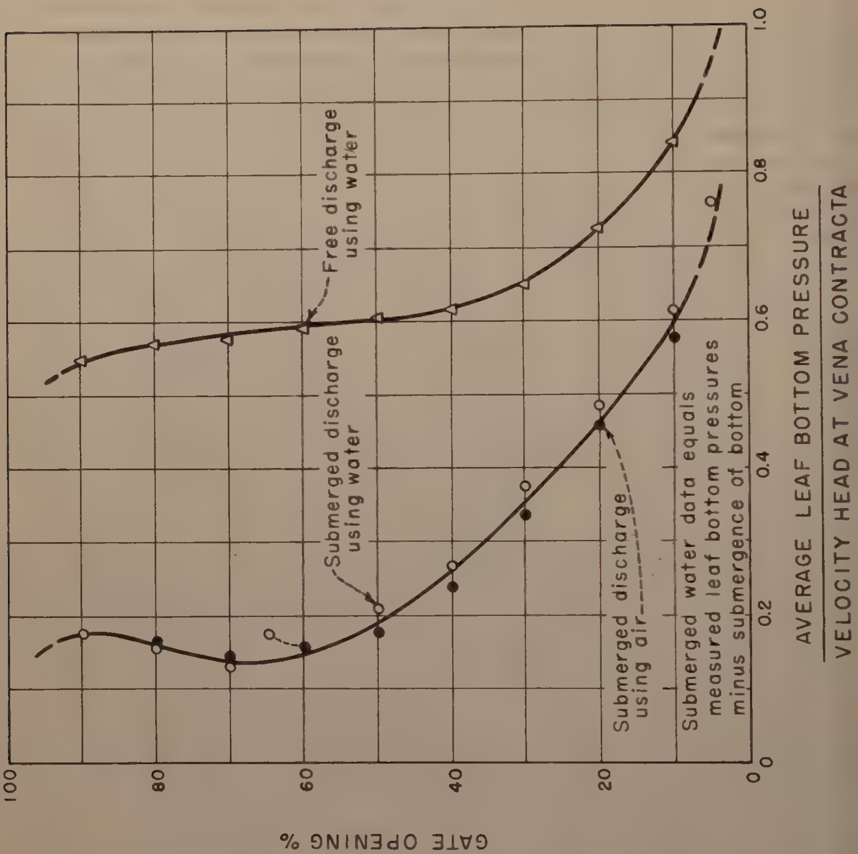
11 "Vertical Hydraulic Forces on Water Controlling Gates," by J. B. Bryce and D. M. Foulds, Proceedings Eighth Congress, IAHR, August, 1959.

12 "Investigations of the Total Pulsating Hydrodynamic Load Acting on Bottom Outlet Sliding Gates and Its Scale Modelling," by A. S. Abelen, Proceedings Eighth Congress, IAHR, August, 1959.



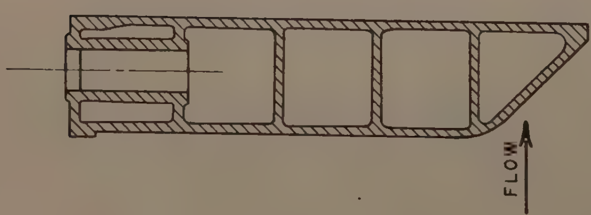




FIG. 1.—GATE LEAF

found by subtracting the upward acting leaf bottom pressures from the downward acting leaf "top" loads and drag forces may not be readily obtained. In the direct measurement method these problems need not be considered because their effects are included in the stem load measurement.

It, thus, appears that there is no single best method for determining, through model studies, the hydraulic downpull forces acting on all high head control gates. Each determination will require careful evaluation of the principal factors contributing to downpull and of the test arrangements most suitable for that structure. In some cases, it may be desirable to use both testing methods.

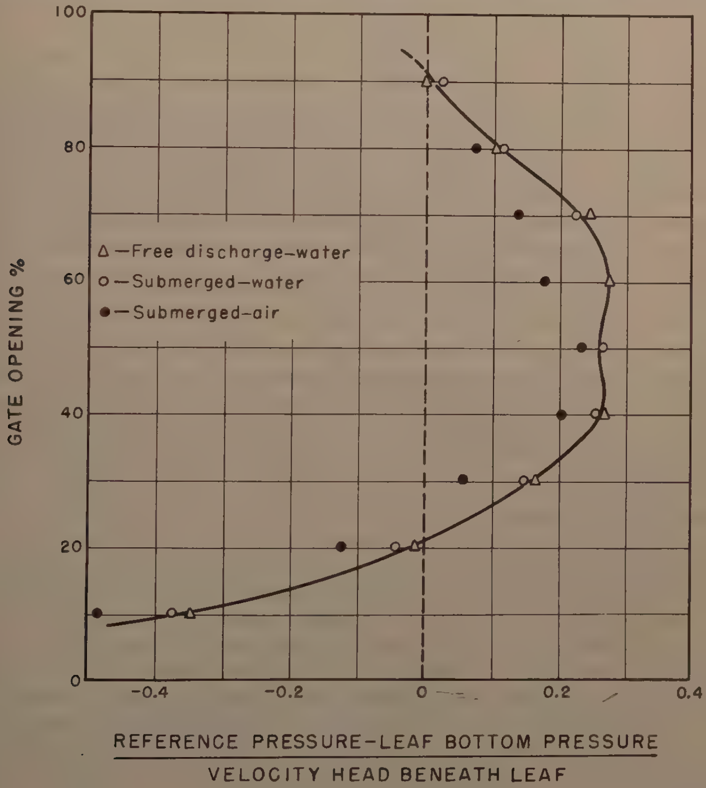



FIG. 3

There are other examples[13] of downpull studies based on the pressure-area method described by the author. These studies were made using air as the flowing fluid instead of water. This is a practice which can result in considerable economies in time, money, and testing equipment. One of the studies concerned a high-pressure, downstream seal, slide gate. The leaf bottom had a 45° sloping surface that was followed by a short, horizontal section that seated on the floor (Fig. 1). The gate was of the type used by the United States Bureau

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13 "Air Model Studies of Hydraulic Downpull on Large Gates," by W. P. Simmons, Jr., Proceedings, ASCE, Vol. 85, No. HY 1, January, 1959.

of Reclamation, Dept. of Interior, at Palisades Dam, Idaho for free-discharge regulation under heads of as much as 237 ft.

The data obtained in the air tests were necessarily for the condition of submerged discharge because the airflow was discharging into a limitless reservoir of like fluid, the atmosphere. However, it was stated that the pressure distribution upon the sloped portion of the leaf bottom would be the same for either free discharge or submerged operation because the flow pattern in this area would remain the same. The pressures themselves would be affected by any submergence on the structure, but the difference in pressure from one station to another on this portion of the leaf would remain unchanged for either type of operation at a given rate of flow.

In the interim between the publication of the writer's paper[13] and the publication of Mr. Colgate's paper, the writer has conducted additional studies with components of the air model. In these later tests, water was used as the flowing fluid so that both free discharge and submerged conditions could be represented. Considerable strengthening and waterproofing were required on the model to enable it to withstand the greater loads imposed by the water and to avoid excessive warpage. To further limit distortion in the model, the tests were conducted in as short a time as possible. A new leaf was used for the water tests in place of the original one, which had become damaged and unserviceable.

The test results were plotted in two different forms. In the first, the measured pressures acting on the leaf bottom, divided by the velocity head at the vena contracta of the flowing water, were plotted against gate opening (Fig. 2). It is readily apparent that the free discharge and submerged discharge values differ greatly, and that the submerged values for these tests are lower than the free discharge ones. The previous air model data are also plotted and show good agreement with the submerged tests using water.

A parameter that better describes the pressure distribution within a flow system is $\frac{p_o - p_x}{h_v}$, in which $p_o$ is a reference pressure ahead of the gate, $p_x$ is the pressure at any point in the system; for instance, on the leaf bottom slope, and $h_v$ is the velocity head of the flow passing beneath the gate leaf. The test data, when expressed in this form, take into account the effect of tailwater because the upstream reference head, which is also directly affected by the tailwater, is used as the datum. The free discharge and submerged discharge data obtained with water tests plot together (Fig. 3). The submerged discharge data obtained with air tests show a similar pattern, but with slightly lower values. Thus, the pressure distributions on the sloped portion of the leaf are shown to be the same for free discharge and submerged operation, and the reasonable agreement of the air and water models is evidence that air models can be used effectively for certain downpull studies.

# DISCHARGE FORMULA FOR STRAIGHT ALLUVIAL CHANNELS[a]

Corrections to Discussion by T. Blench
Discussions by G. H. Lean, Lucien M. Brush, Jr., Don M. Culbertson and Paul R. Jordan, and Bruce R. Colby

CORRECTIONS TO DISCUSSION BY T. BLENCH, F. ASCE.—In the February 1960 Journal of the Hydraulics Division: Page 131. Complete the footnotes at ottom of page as:

. "Regime Behaviour of Canals and Rivers," by T. Blench, Butterworths Scientific Publications, London and Toronto, 1957.

. "The Fourth Root n-f Diagram," by T. Blench, Proceedings, ASCE, Vol. 86, No. HY1, January, 1960.

G. H. LEAN.[1]—The proposed formula rests on data drawn mainly from aboratory flume work, or natural streams which are small enough to rank as lumes. An exception is the data of Lane and Carlson[2] for depths up to 5 ft and ebbles with mean size between 0.8 in. and 3.2 in. With this exception, the epths are seldom greater than 1 ft. Thus, for rippled beds the relative roughess will be large, and the effect of small relative roughness which might be uite important for deep natural streams is not taken into account. So, also, he large-scale sand waves which can occur in deep water[3] and which add apreciably to the roughness are ignored. The general applicability of the index f R and the coefficient $C_a$ is, therefore, in doubt.

The formula for plane beds composed of sand and silt (size 0.06-0.6 mm) s of little direct interest because with long continued action such materials nove in ripples except in certain extreme cases which are seldom found under atural conditions. The evidence that for a rippled bed the velocity for a given epth is not proportional to $S^{1/2}$ is not presented. For the index to be less than (Fig. 10), the ripple roughness must increase (increase in height) as the veocity increases. Flume studies at the United Kingdom's Hydraulic Research tation (HRS) on 0.17 mm sand at well below the critical shear at which ripples isappear[4] do not support this.

It should, perhaps, be stated that the agreement of the calculated and oberved velocities given in Fig. 14 does not prove the formula but verifies the t of the constants of the formula to the data.

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[a] November, 1959, by H. K. Liu and S. Y. Hwang.
[1] Prin. Scientific Officer, Hydr. Research Sta., Wallingford, Berkshire, U. K.
[2] E. W. Lane and E. J. Carlson, Transactions, A. G. U., Vol. 35, 1954, p. 453.
[3] "Systematic Changes in the Beds of Alluvial Rivers," by W. C. Carey and M. D. eller, Proceedings, ASCE, Vol. 83, No. HY 4, August, 1957.
[4] R. A. Bagnold, Philosophical Transactions, Royal Soc., London, A, 1956, 249, 235.

The writer is of the opinion that a field study of the ripple and bed configu ration from which to extract an equivalent Nikuradse roughness is more r liable than the use of a power formula of the type advocated. In this connecti a table of ripple sizes versus equivalent uniform roughness would be inval able.

LUCIEN M. BRUSH, Jr.[5]--Any attempt to establish a discharge formula f alluvial channels is to be commended because of the urgent need of an equati for practical design. Quite justifiably, the authors have used an empiric analysis due to the difficulties they encountered in attempting an analytic approach. However, rational empiricism, by means of dimensional analyse has a sound analytical basis leading to the development of dimensionally h mogeneous expressions. It is often tempting to sacrifice dimensional homogen ity, as the authors have done, for the purpose of simplifying some unwiel expressions, but in so doing, the resulting equation is usually not genera applicable. Thus, the proposed equation, Eq. 31, may be of some use for l boratory flumes, but cannot be extended to prototype alluvial channels, ev as a "first approximation," as suggested by the authors, without extensive fie verification.

The solution of Eq. 31 for the mean velocity requires that the particle si slope, temperature, and the hydraulic radius, with respect to the bed, be know This is somewhat misleading because $R_b$, the hydraulic radius with respect the bed, is not a length parameter which can be measured physically, but actually a conceptual length, the determination of which requires the knowle of the slope, width, depth, velocity (as the authors note), shape of the chann and hydraulic roughness of the side walls. Thus, if $R_b$ is truly known, there only one possible solution for the mean velocity and this solution is complet independent of the empirical curves presented by the authors. In other wor $R_b$ is obtained by applying a wall correction[6,7] to the hydraulic radius. If standard wall correction is valid, only one solution exists for the velocity. plot of computed versus measured velocity would have no scatter as compa to Figs. 13 and 14 shown by the authors. To show that if $R_b$, width, dep slope, temperature, and the shape of the channel are known for a particu flume, only one solution for the velocity exists, it is necessary only to ap the wall correction as outlined by Vanoni and Brooks.[8]

1) Assume a velocity, V.

2) Compute $f_b = \dfrac{8gR_bS}{V^2}$ and $f = \dfrac{8gRS}{V^2}$ in which $f_b$ and f are the resista coefficients for the bed and entire channel, respectively.

3) For smooth walls and a rectangular cross section,

$$f_b = \frac{P}{P_b} f - \frac{P_\omega}{P_b} f_\omega$$

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[6] "Formula for the Transportation of Bed-Load Movement," by H. A. Einstein, Tra actions, ASCE, Vol. 107, 1942.

[7] "The Importance of Considering Side-Wall Friction in Bed-Load Investigation, J. W. Johnson, Civil Engineering, Vol. 12, No. 6, 1942, pp. 329-331.

[8] "Laboratory Studies of Roughness and Suspended Load of Alluvial Streams," A. Vanoni, Report No. E-68, California Inst. of Tech., 1957.

in which $p_b$, $p_w$ and p are the wetted perimeter of the bed, wall, and entire channel, respectively, and $f_w$ is the resistance coefficient for the walls.

4) Compute $\frac{R}{f}$ which equals $\frac{R_w}{f_w}$ in which R and $R_w$ are the Reynolds numbers of the channel and walls, respectively.

5) From a smooth-pipe diagram find $f_w$.

6) Substitute values for $f_b$, f, and $f_w$ in the equation of step 3.

7) Repeat this process until step 3 balances. The resulting velocity is then the unique solution for the given conditions.

For other cross sectional shapes and different types of wall roughness, the procedure is more complicated but will also yield only one solution for the velocity. In requiring that $R_b$, S, d, and t be known, as the authors demand, the solution for the velocity is predetermined and cannot vary unless the wall-correction procedure is assumed to be incorrect.

This result might have been eliminated had the authors chosen either the depth or the hydraulic radius in the final equation (Eq. 31) instead of $R_b$. These parameters can be measured or assumed for different boundary shapes with no knowledge of the mean velocity. Of course, in leaving out the wall correction an equally difficult problem is introduced if data from different flumes are to be compared. This would be unavoidable for the particular approach used.

*Dimensional Analysis*.—Had the authors considered all of the assumptions which they eventually make pertaining to grain shape, sediment uniformity, density variation and supercritical flow during the dimensional analysis, it appears that the computational procedure could have been greatly simplified. More than being a useful tool for efficiently organizing a series of variables, the procedure used in performing a dimensional analysis forces the investigator to examine closely the pertinent variables involved in the problem. Only one of the variables may be dependent, while the others must be independent. If more than one dependent variable is included, superfluous terms will be generated which, if used in graphing the results, may yield correlations which have no bearing on the particular problem being considered. Obviously, the exclusion of an important independent variable will have a more deleterious effect on the analysis and the resulting graphical representations. The authors have chosen to include terms which are superfluous and have omitted some which are of importance.

The variables which the authors feel are important are arranged in the functional relationship,

$$D = \phi\left(q, S, \rho, \mu, g, d, \sigma, \Delta_{\gamma_s}, \eta\right) \qquad (1)$$

The inclusion of $\sigma$ and $\eta$, particularly $\eta$, requires explanation. Sigma $\sigma$, the standard deviation of the particle size distribution may effect the dune height and in so doing affect the depth of flow. However, there seems to be less justification in including the particle-shape factor as an independent variable in determining the depth of flow. Compared to the other variables, the shape factor must be, at best, a second or third-order effect. The authors admit this, of course, because they state that: "In order to improve the accuracy of the method it is necessary to determine the effect of particle shape on the mean velocity." However, the inclusion of $\eta$ allows the authors to substitute fall velocity to their list of variables which already contains particle size. Without

$\eta$, only one or the other of the variables, particle size and fall velocity, may included because of their interdependence (see page 74). Ultimately, both $\sigma$ a $\eta$ are considered to be relatively unimportant by the authors and are exclud from the analysis in the determination of the discharge coefficient $C_a$. Wi the end result being the exclusion of these variables, it is difficult to unde stand why either the particle Reynolds number of the relative-roughness pa rameter should not be excluded from the graphical representations (despi the authors' comments to the contrary), and why the original functional r lationship should not be

$$\phi_a\left(D, S, V, \mu, \rho, g, d, \Delta\gamma_s\right) = 0 \quad \ldots\ldots\ldots\ldots (2$$

or

$$\phi_b\left(D, S, V, \mu, \rho, g, \omega, \Delta\gamma_s\right) = 0 \quad \ldots\ldots\ldots\ldots (2$$

For the assumption of uniform steady flow, which was made by the author slope may be omitted by adding $\tau_0$ if $\gamma$ is substituted for g. Using D, V, a $\rho$ as the repeating variables, the functional relationship for the dimensionle parameters would be

$$\phi_c\left(\frac{\sqrt{\tau_0/\rho}}{V}, \frac{VD}{\mu/\rho}, \frac{V}{\sqrt{\gamma D/\rho}}, \frac{d}{D}, \frac{\Delta\gamma_s D}{\rho V^2}\right) = 0 \quad \ldots\ldots\ldots (3$$

or

$$\phi_d\left(\frac{\sqrt{\tau_0/\rho}}{V}, \frac{VD}{\mu/\rho}, \frac{V}{\sqrt{\gamma D/\rho}}, \frac{\omega}{V}, \frac{\Delta\gamma_s D}{\rho V^2}\right) = 0 \quad \ldots\ldots\ldots (3$$

As the authors have considered only subcritical flow, the third term—t Froude number—may be discarded. However, any important inertial effe are retained in the fifth term. Eliminating the third term, the functional r lationship then reduces to

$$\phi_e\left(\frac{\sqrt{\tau_0/\rho}}{V}, \frac{VD}{\mu/\rho}, \frac{d}{D}, \frac{\Delta\gamma_s D}{\rho V^2}\right) = 0 \quad \ldots\ldots\ldots\ldots (4$$

or

$$\phi_f\left(\frac{\sqrt{\tau_0/\rho}}{V}, \frac{VD}{\mu/\rho}, \frac{\omega}{V}, \frac{\Delta\gamma_s D}{\rho V^2}\right) = 0 \quad \ldots\ldots\ldots\ldots (4$$

Although a complex relationship is still found to exist, the resulting equati 4(a) or (b) are much easier to work with than the extremely complicated K fa tor used by the authors. However, for the assumptions ultimately made by authors, Eqs. 4(a) or (b) are equal in efficiency to the equations which they u Eq. 4(a) suggests that the approach shown in Fig. 2 might yield fruitful resu if the fourth term is included. Eq. 4(b), on the other hand, suggests that Fig might be the starting point for an equally efficient grouping of the basic

ameters provided the additional terms given in Eq. 4(b) are used. The authors, of course, have used Fig. 1 for the purpose of classifying the bed regimes.

As the authors incorporate bed regime in their analysis, it is appropriate to consider briefly another aspect of the problem. A functional relationship for h, the dune height, may be obtained by using an approach similar to that for arriving at Eq. 4(a)

$$\phi_g\left(\frac{\sqrt{\gamma/\rho\, ds}}{V}, \frac{VD}{\mu/\rho}, \frac{d}{h}, \frac{\Delta\gamma_s\, d}{\rho\, V^2}\right) = 0 \qquad (5a)$$

If Eq. 5(a) is combined with Eq. 4(a), after multiplying through by d/D, the following result may be obtained

$$\phi_h\left(\frac{\sqrt{\tau_0/\rho}}{V}, \frac{VD}{\mu/\rho}, \frac{d}{h}, \frac{d}{D}\right) = 0 \qquad (6)$$

Eq. 6 contains two dependent variables each of which is a function of the same three independent variables. Although Eq. 6 can have no cause and effect significance, it may be advantageous to group the data in the form of Eq. 6 for plotting purposes, as is done in plotting a typical lift versus drag-coefficient diagram. For large Reynolds numbers and rather a conservative range in d/D, Laursen obtained[9] a relationship

$$\frac{2D}{h} = f\left(\frac{\sqrt{\tau_0/\rho}}{V}\right) \qquad (7)$$

which is both consistent with the rough-pipe analogy he considered and Eq. 6. The similarity of Eq. 6 to Fig. 2 with an added d/h term is also worthy of note.

Finally, it appears that the general approach used by the authors in attempting to obtain an equation for predicting the velocity of flow in alluvial channel should be pursued farther. Conceptual lengths such as $R_b$, of course, must be eliminated. Bed classification schemes must eventually be replaced by quantitative descriptions of the bed geometry. Dimensional homogeneity should be retained throughout the analysis and resulting equations. However, when these details are worked out, it is conceivable that a useful equation for the prediction of velocity in an alluvial channel can be obtained by using the general outline given by the authors.

DON M. CULBERTSON,[10] M. ASCE and PAUL R. JORDAN.[11]—The proposed discharge formula, although based largely on laboratory flume data representing shallow depths and relatively steep slopes, combines directly or indirectly many of the variables influencing resistance to flow in alluvial channels. The authors are to be congratulated for developing a discharge formula

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[9] "The Total Sediment Load of Streams," by E. M. Laursen, Proceedings, ASCE, Vol. No. HY 1, 1958.

[10] Hydr. Engr., U. S. Geol. Survey, Lincoln, Nebr.

[11] Hydr. Engr. U. S. G. S. Lincoln, Nebr.

based on empirical correlations of the important flow, and sediment parameters and the bed configuration. The formula should be of interest to laboratory workers and may, with additional research, have practical application to channel design and to some special problems encountered on natural streams.

The authors point out that the bed-configuration classification shown in their Fig. 1 is not representative for large natural rivers. They also point out the probable necessity of modifying the discharge formula to suit field conditions. Therefore, this discussion[12] is limited to comparing the bed configuration of selected rivers with that shown in the authors' Fig. 1 and to comparing the field measurements of velocity with those computed from the discharge formula.

Representative examples of the available river data are shown in Fig. 1, in which $V/V_*$ is the third variable. Although these data represent a wide range in depth, slope, mean velocity, water temperature, and sediment discharge, they represent a narrow range in median particle size of bed material. All the rivers considered are reasonably straight at the measuring sections; bank friction probably is insignificant because the depth-width ratios are low. Because the bed configuration is usually difficult to observe in flow more than about 3 ft deep and because accurate measurements of bed configuration for the Middle Loup River only are available to the writers, the ratios of $V/V_*$ are used to indicate the bed configuration. Low ratios of $V/V_*$ (about 6 to 11 for the Cedar, Middle Loup, Niobrara, and Little Blue Rivers and about 10 to for the Mississippi River at St. Louis) indicate dune bed. High ratios of $V/V_*$ (about 15 to 18 for the Middle Loup River and about 18 to 23 for the Mississippi River) indicate transition or flat bed. Intermediate ratios of $V/V_*$ indicate dunes and transition in different parts of the cross section and are not plotted on Fig. 1. It is apparent from Fig. 1 that the parameters $V_*/w$ and $wd/\nu$ are inadequate to estimate the bed configuration of streams such as the Middle Loup River, however, they may be adequate for laboratory flumes and for some rivers. For median bed-material sizes of 0.2 mm to 0.5 mm the river data indicate that the average curve of the authors' Fig. 1 for transition bed is too low.

According to the authors' Fig. 1, a given shear velocity will produce the same bed configuration for a fairly wide range of median particle sizes, except in the region near the beginning of sediment motion and ripple formation. For example, at a constant temperature of 65° F and a median particle size of 0.20 mm, a shear velocity of about 0.18 fps is required to maintain a transition bed. If the particle size is 0.80 mm, the required shear velocity is still about 0.18 fps.

Data from the field measurements seem to confirm the authors' Fig. 7. Although these data generally are outside the range of K values of the laboratory data, they are in good agreement with the related lines of constant $wd/\nu$. The agreement was further apparent on the authors' Fig. 8. Agreement was good even when the bed did not have the appearance of a dune bed and the roughness, as indicated by Manning's n or Chezy's C, was much less than is common for dune beds. The excellent agreement may be due partly to the similiarity of the parameters $V_*d/\nu$ and K. The small deviation of both laboratory and field data from the curve of Fig. 8 indicate that Eq. 31 should give very good results when $C_a$ is obtained from Eq. 32. However, Eq. 32 contains

$$\left(\frac{1}{\Omega\sqrt{A}}\right)^{\frac{1}{1-n}}$$

[12] Publication authorized by Director, U. S. Geological Survey.

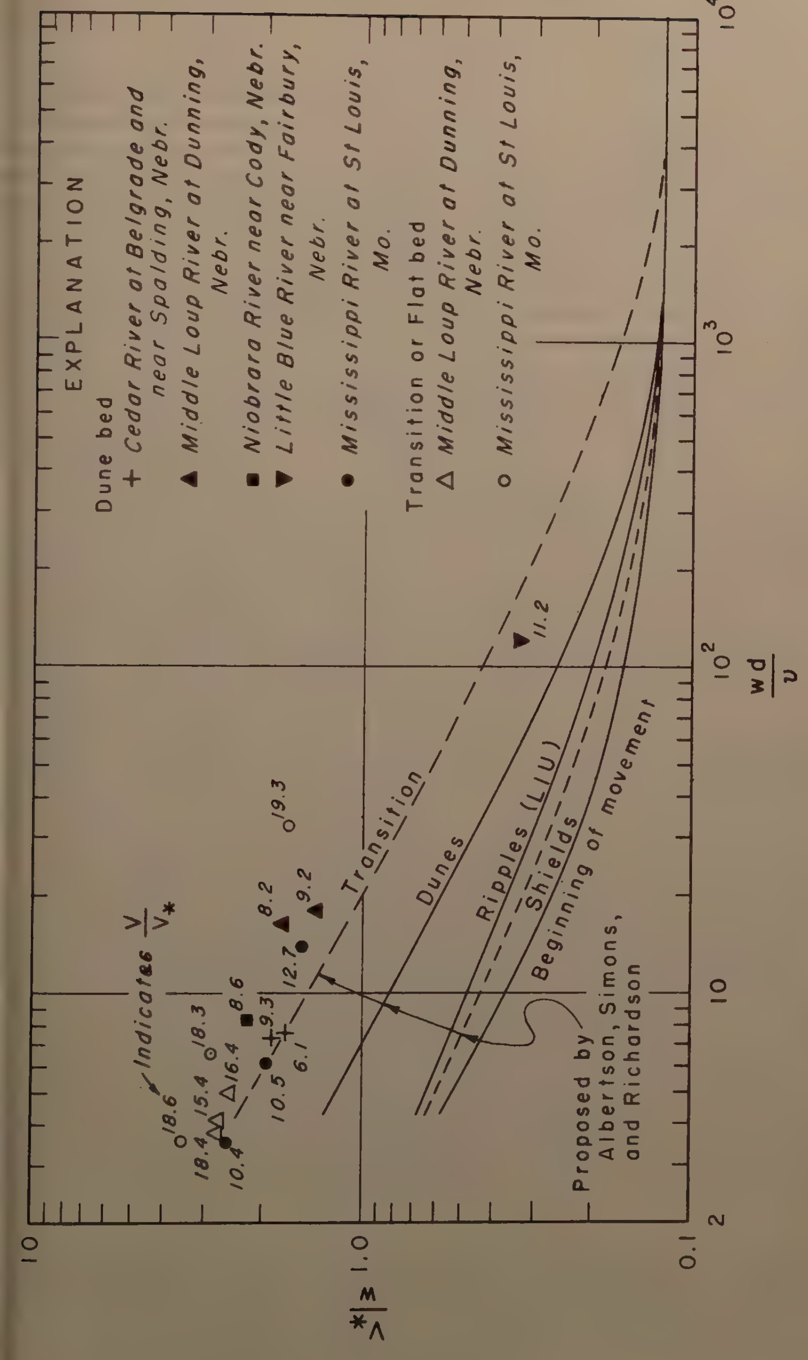


FIG. 1.—CLASSIFICATION OF THE CONFIGURATION OF ALLUVIAL BED

which for dune bed is equivalent to $A^{-5.9}$. Note that a 5% error in A can result in about a 35% error in $C_a$. Thus, the computation of $C_a$ from Eq. 32 not only is tedious but can result in large errors.

Velocities computed with the coefficients and exponents for dune bed are generally within about 20% of the measured velocities, whereas velocities computed with the coefficients and exponents for flat bed are nearly always much higher than the measured velocities. Table 1 shows the pertinent data for two rivers at widely different measured velocities. The bed configuration for the Middle Loup River near Dunning, Nebr., was determined with an ultrasonic sounder, and the bed configuration for the Mississippi River at St. Louis, Mo., was assumed on the basis of $V/V_*$.

The data in Table 1 are typical of many field measurements; computed velocities for flow over a flat bed are unreasonably high. Also, velocities computed with Eq. 31 may be unrealistic for alluvial rivers having small slopes

TABLE 1.—DATA FROM FIELD MEASUREMENTS

| | Middle Loup River at Dunning, Nebr. | | Mississippi River at St. Louis, Mo. | |
|---|---|---|---|---|
| | June 26, 1959 | Dec. 6, 1959 | Feb. 21, 1956 | Apr. 30, 195? |
| Bed configuration | Dune[a] | Flat | Dune | Flat |
| Median particle size of bed material. mm. | 0.31 | 0.29 | 0.31 | 0.48 |
| Water temperature degrees F | 79 | 35 | 36 | 65 |
| Slope | .00104 | .00118 | .0000822 | .000110 |
| Hydraulic radius foot | 2.15 | 1.52 | 16.5 | 44.7 |
| Shear velocity feet per second | .268 | .240 | .209 | .398 |
| $V/V_*$ | 8.2 | 16.4 | 10.5 | 19.3 |
| Measured velocity feet per second | 2.20 | 3.95 | 2.20 | 7.69 |
| Computed velocity: | | | | |
| Flat bed feet per second | | 6.05 | | 17.2 |
| Transition (ripples bars) feet per second | | 2.90 | | 9.54 |
| Dunes feet per second | 1.86 | | 2.13 | |

[a] Average amplitude 0.72 ft, average length 12.5 ft.

shallow depths, and bed material of coarse sand. For example, the compute velocity of flow over a dune bed is higher than the computed velocity over a fla bed when the slope, depth, and median particle size of bed material of the Littl Blue River are 0.0005, 3.0 ft, and 1.00 mm, respectively.

In summary, a comparison of the observed bed configuration of selecte rivers with the classification of the configuration of alluvial beds, according t the authors' criteria, suggests that the authors' Fig. 1 is inaccurate in th ranges of dune bed and transition. Velocities computed with the authors' discharge formula were much higher than the measured velocity for flow over flat bed but agreed reasonably well for flow over a dune bed.

BRUCE R. COLBY.[13]—The proposed discharge formula, which is here assumed to mean the authors' Eq. 31 and Figs. 1, 9, 10, and 11, shows clearl

[13] Hydr. Engr., US Geo. Survey, Lincoln, Nebr.

he wide range of resistance to flow for different bed configurations. It im-
plies, by the omission of sediment concentration from the formula, that any major effect of sediment concentration on mean velocity correlates with bed configuration, fall velocity of the median particle, or $R_b^x S^y$. The authors are o be commended for bringing out these points.

Presumably the discharge formula is intended for use in natural streams vithout extensive modification, although it was derived from flume experiments nd accuracy was not claimed for the formula when applied outside the labo-atory. The purpose of this discussion is to explain some of the difficulties of sing the formula for a stream outside the laboratory.

One disadvantage of the formula is that it was developed for shallower depths nd higher slopes than those of most natural streams. Hence, the extension of he formula to deeper flows and flatter slopes may lead to inconsistent results. 'or example, the computed mean velocity is slightly higher over a dune bed han over a flat plane bed for a median diameter of 1 mm, hydraulic radius of ft, and slope of 0.0006. (Fig. 1, as will be indicated later, is not accurate nough to show which bed configuration would actually exist for these con-itions.)

Other disadvantages of the proposed discharge formula become apparent hen it is used to compute the depth-discharge relationship for Pigeon Roost 'reek near Byhalia, Miss. The median diameter of the bed material is about .4 mm, and the surface slope at steady flow is about 0.0011. Near the gaging tation, the flow is in a dredged channel about 80 ft wide and straight for many undred feet both upstream and downstream. A water temperature of 16° C was ssumed for the computations. Discharges per foot of stream width were based irectly on Figs. 1, 9, 10, and 11 and are plotted on Fig. 2 of this discussion s a solid line that has horizontal jogs at changes from one bed configuration another.

Streamflow measurements on Pigeon Roost Creek, near Byhalia, define two urves of stage-discharge relationship. One of the curves is for flow over a une bed and one for flow over the field equivalent of a flat bed. The curves re expressed on Fig. 2 as depth-discharge relationships per foot of stream idth and are applicable for steady flow. Manning's equation with n's of 0.032 nd 0.016 can be used to reproduce rather closely the curves for the low-velocity urve and the high-velocity curve, respectively. The curve for high-velocity ow has no break even at depths when antidunes sometimes occur along the iddle of the channel. Some streamflow measurements, made when the dis-harge was rapidly changing and depths were within the approximate range of ft to 3 ft, plotted between the two curves.

Several conclusions can be drawn by comparing discharges that are com-ted from the proposed discharge formula with discharges that are based on reamflow measurements. First, the agreement between computed and mea-red flows over a dune bed is very good. However, Fig. 1 incorrectly indi-tes that dunes should not exist at depths greater than 0.9 ft for the median rticle size, slope, and water temperature that were used, whereas dunes arly always exist up to depths of 2 ft and may exist at depths greater than ft.

Second, over a flat bed the discharges computed from the formula are ap-eciably higher than the measured discharges. Perhaps a flat bed in Pigeon ost Creek is less regular than a flat bed in a laboratory flume. Also, bank ction may have an appreciable effect at depths greater than 4 ft, but its ef-ct probably is insignificant at depths of 2 ft to 3 ft. A reasonable tentative

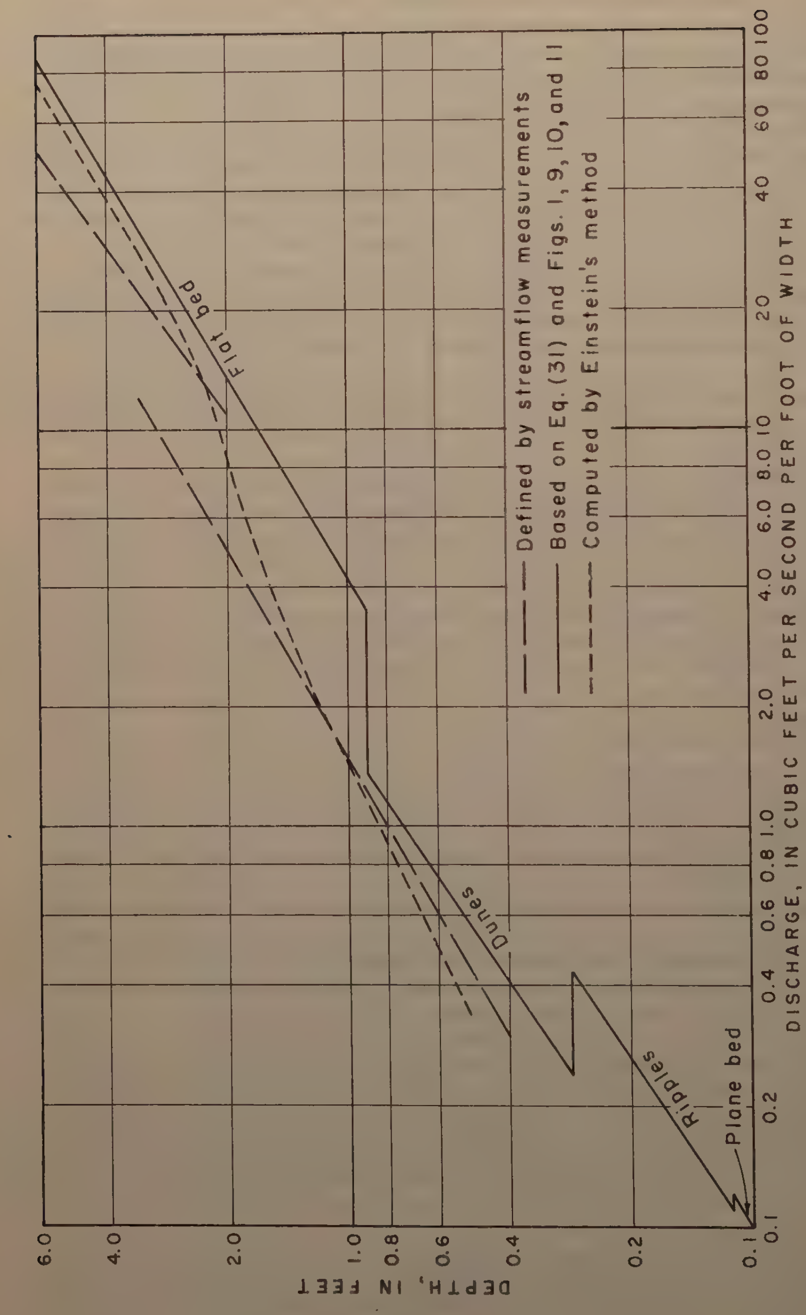


FIG. 2.—COMPARISON OF MEASURED AND COMPUTED DISCHARGES OF PIGEON ROOST CREEK NEAR BYHALIA, MISS.

conclusion is that the formula, generally, may give somewhat too high discharges for flow over a flat bed in a natural stream.

Third, Fig. 1 gives too low a shear velocity for the change from a dune bed to a flat bed. The other changes in bed configuration are at flows so low that neither the changes nor the velocities have much practical significance in many natural streams.

Incidentally, Fig. 1 does not indicate the limiting shear velocity for the change from dunes to bars and a direct change from dunes to a flat bed has been assumed. Bars and other bed configurations intermediate between dunes and a flat bed are relatively insignificant for Pigeon Roost Creek near Byhalia, partly for reasons that are associated with length of upstream channel. Consider a reasonably uniform alluvial channel several miles long at whose upper end the flow is being increased slowly. When the flow becomes slightly too great to be transported over a dune bed, the velocity increases considerably even though the discharge increases only a little. Hence, the water at the higher velocity tends to overtake the slower water ahead of it. As a result, the rising side of the hydrograph of flow becomes increasingly steep as the flow moves downstream.

After the water has traveled a few miles, the flow at the change from a dune bed to a flatter bed no longer increases gradually but may double within a few minutes. The rapid increase to a relatively high discharge causes an abrupt change from dune configuration to flat bed. On the recession side of the hydrograph, the flow at some point upstream may be decreasing gradually when the flat bed begins to change to bars or dunes. The reduction in velocity causes the channel to drain less rapidly. As the flow progresses downstream, the recession side of the hydrograph of flow becomes increasingly steep at the time of change from a flat bed to a rougher bed. This faster decrease of flow makes the change from the flat bed to a dune bed more rapid than it might have been upstream where the decrease of flow was more gradual. In other words, gradually changing rates of discharge may be unlikely, if not impossible, at the change from a dune bed to a flat bed or from a flat bed to a dune bed if a long and reasonably uniform alluvial channel exists upstream.

Fourth, comparison of the computed depth-discharge relationships with those that are defined by streamflow measurements indicates (Fig. 2) that the shape of the transitions from one bed configuration to another are unrealistic. At the transitions from plane bed to rippled bed and from rippled bed to dune bed, the discharge formula and the authors' Fig. 1 indicate that a slight increase in depth will reduce the mean velocity. (The computed reduction in velocity at the change from a plane bed to a rippled bed is thought to be too small.) If such a reduction in velocity actually occurred and was of appreciable amount, the bed configuration probably would return to the form that has a lower resistance to flow.

The shape of the transition of computed discharges from a dune bed to a flat bed is, also, incorrectly shown by a single horizontal line. According to the computed discharges on Fig. 2, steady flows of 1.4 to 3.4 cfs per ft cannot occur. That is, both the velocity and the computed discharge are discontinuous at the change from a dune bed to a flat bed. Measured velocities are, indeed, discontinuous for steady flows at the change from a dune bed to a flat bed, but measured discharges for steady flows have no appreciable discontinuity because the depth of flow can readily adjust to the bed configuration and the discharge. Thus, the depth-discharge curves defined by streamflow measure-

ments overlap in depth of flow. (See Fig. 2). Within the range of overlap they supply field verification of the conclusion[14] by Brooks that the velocity cannot be "expressed as a single-valued function of the bed shear stress, or any combination of depth and slope, or bed hydraulic radius and slope." In this range of about 1 ft of depth on Pigeon Roost Creek near Byhalia, neither depth nor shear velocity can indicate whether the bed will be flat or be covered with dunes. This is a fundamental weakness of any relationship such as that of the authors' Fig. 1 that makes bed configuration a function of no flow parameter except shear velocity.

Overlapping of the depth-discharge curves for a dune bed and for a flat bed is somewhat unusual in natural streams. It requires that the bed be almost entirely covered by dunes at some times and by a flat bed at the same depth of flow at other times. Such a consistently uniform bed configuration of one type or the other in the range of depth for which either a dune bed or a flat bed might exist seems to be possible only if the lateral distributions of depth, velocity, and bed-material size are fairly uniform.

Most streams have irregular lateral distributions of depth, velocity, and bed-material size and hence part of the bed may have one type of configuration while another part has another type. Also, part of the bed may have some intermediate type of bed configuration. The alluvial streams that have no clear-cut discontinuity in the depth-discharge relationship generally have a range of depth at which the resistance to flow varies widely and somewhat uncertainly. For those streams having a reasonably average transition of bed roughness from dune configuration to flat bed, the method used[15] by Einstein for computing mean velocity probably gives reasonably accurate discharges.

Even for Pigeon Roose Creek near Byhalia, the discharges computed by the Einstein method (Fig. 2) are somewhat more accurate than those computed by the authors' discharge formula. The depth at which the transition between the two types of bed configuration occurs is also better determined from the Einstein method. However, for depths of 1.5 ft to 3.0 ft the discharges computed with the Einstein method are considerably different than those that are determined by streamflow measurements.

As compared to the flow of Pigeon Roost Creek near Byhalia, the authors' discharge formula gives computed discharges that agree well with measured flows over a dune bed but are too high for flow over a flat bed. Also, the authors' Fig. 1 is an inadequate measure of the range of depths at which transitions occur between dune beds and flat beds or vice versa.

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14 "Mechanics of Streams with Movable Beds of Fine Sand," by Norman H. Brooks, Transactions, ASCE, Vol. 123, 1958, p. 547.

15 "The Bed-Load Function for Sediment Transportation in Open Channel Flows," by H. A. Einstein, Tech. Bulletin No. 1026, U. S. Dept. of Agric., Washington, D. C., September, 1950.

# EFFECT OF AQUIFER TURBULENCE ON WELL DRAWDOWN[a]

Discussion by J. R. Philip

J. R. PHILIP.[1]—I wish to comment on only one aspect of this paper, namely the view implicit in it that "in laminar flow, the energy losses are directly proportional to the velocity," and that turbulence is a term embracing all flows "under conditions that relate the energy loss to a velocity exponent that is greater than one."

In fluid mechanics, the terms "laminar" and "turbulent" have quite definite meanings, and I feel that we must not ignore these meanings and set up new and quite different ones. Although some profound questions about turbulence remain very much unsettled[2] we can quite definitely assert that flows may be laminar, in the sense of being ordered flows[3] that obey the Navier-Stokes equation, without there being a linear relation between flow rate and energy loss. As the writer shall outline here, there is very strong evidence that flows in porous media in the Reynolds number range of 10 to 100 or over (these numbers depend somewhat on how Reynolds number is defined) belong to this laminar, but non-linear, regime. The confusion on this point seems to have arisen from false analogies between flow in tubes and flow in porous media. Unfortunately, these misleading ideas on "turbulence" in porous materials appear in the text books on flow in porous media.

The steady state form of the Navier-Stokes equation for an incompressible fluid is

$$- U \times (\nabla \times U) + \frac{1}{2}\nabla U^2 = - \nabla S + a\nabla^2 U \qquad (1)$$

in which U is the (microscopic) vector velocity, and S is the potential defined by

$$S = P\text{-}d + Q. \qquad (2)$$

in which P is the pressure, d the fluid density and Q the potential of the external forces. (Note that $\frac{1}{2}U^2$, P/d and Q are, in a gravitational field, entirely analogous to the velocity, pressure and gravitational components of "head" in hydraulic engineering.)

Textbooks give[3] the derivation of Eq. 1 from simple considerations about the mechanics of a fluid in motion. The forces and accelerations on a small element of the fluid are related on the supposition (rigorous for Newtonian fluids, such as water) that the shearing stress sustained by the fluid is proportional to

[a] November, 1959, by Joe L. Mogg.

[1] Prin. Research Officer, Div. of Plant Industry, C.S.I.R.O., Canberra, Australia.

[2] For example, with stream surfaces constant in time. "International Symposium on Atmospheric Turbulence in the Boundary Layer," Geophys. Res. Papers, 19, pp. 479-491, 1952.

[3] "Hydrodynamics," by H. Lamb, 6th Ed. Cambridge University Press, p. 576, 1952.

the velocity gradient normal to the plane of shear. The (constant) ratio of shearing stress to velocity gradient is the dynamic viscosity of the fluid, b, which is equal to da.

Navier obtained Eq. 1 as early as 1822, the theoretical basis being improved by Saint-Venant in 1843. Curiously enough, this general analysis of the motion of a viscous fluid preceded the much less general, but better known, studies of Poiseuille and Darcy. It does not seem to have been recognized at the time that their results represented simple cases of Eq. 1.

The terms on the left hand side of Eq. 1 are known as the inertia terms. These terms are identically zero for certain special motions including axial flow in straight tubes. In this case Eq. 1 reduces to

$$\nabla S = a\nabla^2 U. \quad (3$$

It follows that, for such cases the flow rate is directly proportional to the energy losses until the flow is increased to the point where the orderly flows known as "laminar" give way to disorderly unsteady "turbulent" flow. As is well known[4] this occurs for straight tubes of circular cross section at Reynolds numbers of the order of 2,000.

The difference in character between flow in tubes and in porous media is, in mathematical terms, that in porous media the inertia terms are not identically zero, and that the linear regime in porous media depends on the approximation that the inertia terms may be neglected. In fact, it can be shown[5] that this approximation can be expected to fail when the Reynolds number is unity or perhaps a little more (depending again on the definition used). This failure of the linear (Darcy) regime in porous media at flows in Reynolds number range, from just over unity to 10 or 20 times this, is well known experimentally.[6,7] The fact that the non-linearity arises where it is predicted by the Navier-Stokes equation is very cogent evidence that it has its origins in the non-vanishing of the inertia terms and not in the onset of turbulence. (Of course if the flow rate were continuously increased, a point would ultimately be reached where a genuine turbulent regime had been established.)

Physically, this difference in character between laminar flow in tubes and in porous media is not at all surprising, when we reflect on the fate of a small parcel of fluid moving through each system. In the tube the parcel possesses a constant linear velocity throughout, while in the medium it is subject to forces which continuously change both the magnitude and direction of its velocity.

The Swedish engineer Lindquist[8] recognized the central role that the inertia terms play in producing the non-linear laminar regime in 1932. Muska questioned[9] this interpretation because diffusion of marker solution increase

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4 "Modern Developments in Fluid Mechanics," by S. Goldstein, Clarendon Press, Oxford, Vol. 1, 1938, p. 319.

5 "Hydrodynamics," by H. Lamb, 6th Ed. Cambridge University Press, p. 599, 195

6 "The Flow of Homogeneous Fluids through Porous Media," by M. Muskat, McGraw Hill Book Co., Inc., New York, 1937, p. 56.

7 "The Physics of Flow through Porous Media," by A. Scheidegger, Univ. of Toron Press, 1957, p. 124.

8 "On the Flow of Water through Porous Soil," by E. Lindquist, 1er Congres de Grands Barrages, Stockholm, 5, 1933, pp. 91-101.

9 "The Flow of Homogeneous Fluid through Porous Media," by M. Muskat, McGra Hill Book Co., Inc., New York, 1937, p. 62.

with flow velocity; but the phenomenon of "hydrodynamic dispersal"[10] is now[11] understood and occurs in the absence of turbulence.

Finally, we note that recent mathematical work by Tamada and Fujikawa[12] on the drag on arrays of cylinders and other obstacles gives a very graphic picture of the way in which the non-linear regime develops as the Reynolds number increases and the inertia terms grow in importance.

These matters have been treated further elsewhere.[13] Irmay discusses[14] the non-linear laminar regime in a different, though closely connected way.

Summarizing, then, this note advocates (and advances reasons for) the recognition of three regimes of flow in porous media. These are, in order of increasing Reynolds number: (a) Linear laminar (or Darcy) regime, (b) Non-linear laminar regime, and (c) Turbulent regime. The paper under discussion, and other references in the literature, misleadingly label item (b) "turbulent."

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10 Due to diffusion across stream surfaces and to disruption of the stream system at each stagnation point in a porous medium.

11 "Dispersion of a Moving Salt-Water Boundary Advancing Through Saturated Sand," by P. R. Day, TAGU, 37, 1956, pp. 595-601.

12 "The Steady Two-Dimensional Flow of Viscous Fluid at Low Reynolds Numbers Passing through an Infinite Row of Equal Parallel Circular Cylinders," by K. Tamada, Quart. Jour. Mech. Appl. Math., 10, 1957, pp. 425-432.

13 "Physics of Water Movement in Porous Solids," by J. R. Philip, Highway Research Board, Washington, Special Report, 40, 1958, pp. 147-163.

14 "On the theoretical derivations of Darcy and Forchheimer formulas," by S. Irmay, TAGU, 39, 1958, pp. 702-707.

# GENERALIZED DISTRIBUTION NETWORK HEAD LOSS CHARACTERISTICS[a]

Discussion by E. F. Trunk

E. F. TRUNK,[1] F. ASCE.--While the method described has been used and found to be applicable to gas distribution networks, it is felt to be more limited in its application for several reasons. The chief reasons being the differences in the character of the system load in the gas industry as compared to the water industry, and the differences in gas network design by reason of dealing with a compressible fluid.

The widespread use of gas for space heating produces a large swing in load. It is not unusual to find the maximum day in the winter to be five or six times the average summer base load. The load pattern in a gas system is seldom a smooth swing from maximum day to summer base load. It is broken in several places when blocks of interruptible customers are permitted to come on the line, thus changing the flow pattern in the entire network. Hence, a comparison of pressure losses by the method outlined would be limited to comparing loads occurring in the same system flow pattern.

Due to the compressible nature of the fluid handled, gas distribution systems are often supplied at numerous points by district regulators that, in turn, are supplied by a system operating at a higher pressure level. This, again, limits the application of the method outlined to smaller segments of the distribution network.

Even with the limits mentioned, the writer has found the method to be valuable and a time-saving tool in making rapid calculations of the load-carrying capacity of sections of a gas distribution system. It is particularly valuable to the load dispatcher who is quite often faced with making a quick estimate of pressure drops without having enough time for detailed computation or an analyzer set-up.

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[a] January, 1960, by M. B. McPherson.
[1] Chf. Engr., Laclede Gas Co. St. Louis, Mo.

# THE FOURTH ROOT n-f DIAGRAM[a]

## Corrections

CORRECTIONS.—Page 64, INTRODUCTION, third paragraph. In line 2 omit comma after "chart." In line 3 replace "at" by "to." Fourth paragraph, in line 5 omit semicolon.

Page 66, first paragraph. In line 14 insert "he" between "but" and "did." In line 20 replace "is" by "we." In line 21 replace "sociated" by "sociate." In line 25 replace "curvilinear" by "logarithmic."

Page 68. In line 2 replace the references "3,4" by "8,10."

Page 69. In section iv, line 10 insert "it" before "will."

Page 72. In line 6 after Eq. 4 insert "n" before "should."

Page 73. In subsection i, line 3, insert "square root of" before "Reynold's Number."

Page 72. In the equation preceding Eq. 3, insert missing bracket.

[a] January 1960, by T. Blench.

# BOUNDARY LAYER STIMULATION IN RECTANGULAR CONDUITS[a]

## Corrections

CORRECTIONS.—In the February Journal of the Hydraulics Division, on age 29, please delete the superscript 4 that appears in the second line above ig. 4, and add a subscript * to the delta in that same line. Also, on the second ine below Fig. 4, please delete the superscript 4, and in Eq. 2 please add a ubscript * to the delta that precedes the equals sign. On page 35 please interhange the illustrative material that is presently identified as Fig. 12 with that hich is presently identified as Fig. 13. That is, the captions remain in their resent relative position on the page but the drawings are moved.

[a] February, 1960, by R. G. Cox and F. L. Bauer.

# TRAP EFFICIENCY OF RESERVOIRS, DEBRIS BASINS, AND DEBRIS DAMS[a]

## Discussion by Rolland F. Kaser

ROLLAND F. KASER,[16] F. ASCE.--The data on trap efficiencies of reservoirs presented by the authors will certainly be helpful to engineers who must plan similar reservoir projects. Some major dissimilarities between reservoirs cited by the authors have not been mentioned, however, and the facts should be available to those considering use of the data. The writer has not examined the circumstances peculiar to all of the facilities listed in the author's Table 1, but those concerning the Colorado River facilities, with which he is familiar, will serve to illustrate the point.

The Imperial Dam reservoir, at Yuma, Ariz., is listed as a "Normal Ponded Reservoir," in Table 1, with a drainage area of 184,000 sq miles, and data are given for two 5-yr periods of operation (1938-1942 and 1943-1947). As illustrated by the map on Fig. 7, Imperial Dam is situated downstream from Hoover Dam (drainage area 167,800 sq miles; closure to form Lake Mead in February, 1935); Davis Dam (controlling 1,520 sq miles downstream from Hoover Dam; closure to form Lake Mohave in 1950); and Parker Dam (controlling 9,500 sq miles downstream from Davis Dam; closure to form Havasu Lake in October, 1938). Imperial Dam was closed in April, 1938. The drainage area which contributes sediment to the reservoir formed by Imperial Dam is limited to the 5,800 sq miles downstream from Parker Dam.

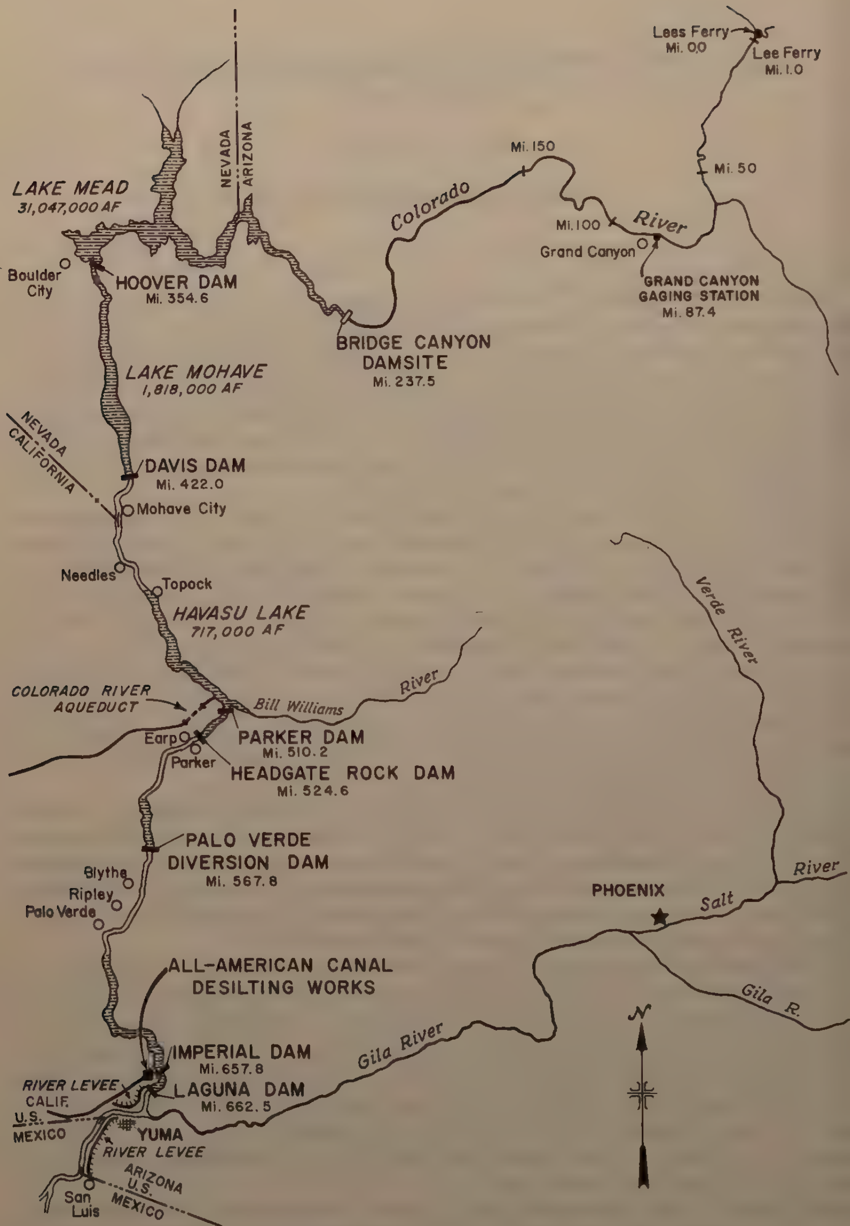
Inflow to Imperial Dam Reservoir, however, is almost entirely up-river runoff released through Parker Dam. The sediment-free water, which has been released through the gates or turbines at Parker Dam, picks up a new sediment load from the bed and banks of the Colorado River downstream from Headgate Rock Dam. It was this coarse river-bed material, from nearly 100 miles of degrading channel, that filled Imperial Dam Reservoir.

Throughout the 10-yr period cited (1938-1947), the lower Colorado River was adjusting to a new regime brought about by construction and operation of new dams, reservoirs and diversion structures. Under these circumstances, the C/W and C/I ratios cannot be expected to bear the same relationships to sediment production and trap efficiency as they would for a reservoir in the same watershed which received unregulated inflows. It is noted that data for Imperial Dam Reservoir on Fig. 1 plot far to the left of the curve. It would, of course, be only by coincidence that data for such a reservoir would line up with data for "normal" reservoirs.

The authors have presented specific data for Imperial Dam Reservoir on Fig. 2. It is unfortunate that the peculiar conditions that apply were not emphasized. The writer has called attention to the source of sediment in the lower

[a] February, 1960, by Charlie M. Moore, Walter J. Wood and Graham W. Renfro.

[16] Chf. Planning Engr., U. S. Study Commission-Texas, 980 M and M Bldg., Houston, Tex.



Lees Ferry
Mi. 0.0
Lee Ferry
Mi. 1.0
NEVADA
ARIZONA
Mi. 150
Mi. 50
LAKE MEAD
31,047,000 AF
Colorado
Mi. 100
River
Grand Canyon
Boulder
City
HOOVER DAM
Mi. 354.6
GRAND CANYON
GAGING STATION
Mi. 87.4
BRIDGE CANYON
DAMSITE
Mi. 237.5
LAKE MOHAVE
1,818,000 AF
NEVADA
CALIFORNIA
DAVIS DAM
Mi. 422.0
Mohave City
Needles
Topock
Verde River
HAVASU LAKE
717,000 AF
COLORADO RIVER
AQUEDUCT
River
Bill Williams
Earp
PARKER DAM
Mi. 510.2
Parker
HEADGATE ROCK DAM
Mi. 524.6
PALO VERDE
DIVERSION DAM
Mi. 567.8
Blythe
Ripley
Palo Verde
River
PHOENIX
Salt
ALL-AMERICAN CANAL
DESILTING WORKS
Gila R.
N
IMPERIAL DAM
Mi. 657.8
Gila River
RIVER LEVEE
CALIF.
LAGUNA DAM
Mi. 662.5
U.S.
MEXICO
YUMA
RIVER LEVEE
ARIZONA
U.S.
San
Luis
MEXICO


FIG. 7

Colorado River that explains the coarse nature of the material transported into the reservoir. Such sediments would have a different relationship between detention time and trap efficiency than would apply to other particle sizes. Another factor influencing the trap efficiency is the practice of weekly or bi-weekly sluicing through the California sluiceway, at Imperial Dam, to remove sediment deposits, that might interfere with diversions to the All-American Canal, and to remove bars in the sluiceway channel formed by sludge pipe discharges from the All-American Canal Desilting Works. This practice, that was in effect during the period of record cited, involved, in sequence, (1) closure of the canal headgates, (2) complete opening of the sluicegates, (3) complete drawdown of the reservoir, (4) closure of the sluicegates until the normal reservoir pool level was restored, and (5) opening of the canal headgates for normal diversions. Extra water was released from Parker Dam, in synchronization with the sluicing operation, to minimize the time required to refill the reservoir so that the canal shutdown period would be as short as possible. The authors have not explained how the sediment data in their Table 1 were computed, but there is nothing to indicate that the above factors were taken into account. In any event, it is quite unlikely that the relationship between trap efficiency and detention time, shown by Fig. 2, would apply to any other reservoir.

The foregoing comments and explanation also apply to the All-American Canal Desilting Basins to some extent. Certainly, the size of the drainage area upstream from Imperial Dam (as listed in column 5 of Table 1 for record No. 38) had little influence on the sediment entering the All-American Canal during the period of record cited.

The authors have presented, in column 7 of Table 1, figures for the average annual sediment production per square mile of drainage area and refer readers to their reference paper[4] by Brune, for details. It is interesting to note that a rate of 2,170 tons per sq mile is given for Imperial Dam Reservoir, while only ,044 and 406 tons per sq mile are given for Lake Mead and the All-American Canal Desilting Basins, respectively. Obviously the rates given cannot all be applied to the drainage areas listed in column 5, and it will be most helpful to anyone interested in the data in Table 1 if the authors will explain the derivation of the figures and the limitations on their use.

It appears from the data presented by the authors that so long as the C/I ratio exceeds about 0.3, planners would be justified in assuming that all of the sediment inflow would be deposited and that trap efficiency is a matter requiring special study only on ponds and relatively minor reservoirs.

# SCOUR AT BRIDGE CROSSINGS[a]

Discussion by T. Blench

T. BLENCH,[9] F. ASCE.—The writer draws attention to the collection of data of scour around bridge piers and other obstacles made by C. C. Inglis, from 1924 to 1942.[10] The data of severe scour around piers numbered 17 and were for floods from 30,000 cfs to 2,600,000 cfs. They appeared, with descriptions, as Table 8-1 in the reference and were plotted in its Fig. 8-1 to show the relation of scour depth (measured from water surface to bottom of scour) to the Lacey regime depth for the river at the flood discharge; the line in Fig. 8-1 was an excellent fit and indicated that, all other things being equal, scour depth varies as the cube root of discharge exactly as the regime theory based on controlled canals shows; the ratio to regime depth averaged almost exactly 2.0. The writer has replotted[11] the Inglis data along with model data of Inglis, of the present author (some time ago), and of workers at the University of Alberta. The coordinates were chosen in terms of estimated discharge intensity for the sake of uniformity and future applications; allowing for the discharge intensities of the rivers being averages over the whole river whereas for models they are discharge intensities of attack, the river and model data were in good accord with each other and with the regime theory prediction of variation as the two thirds power of discharge intensity—all other factors being equal. What to do when other factors vary has been explained.[11]

Inglis has described,[10] also, experiments on model bridge piers, similar to the recent Iowa ones, has plotted the results, and has given a formula to fit the graph. The writer has reduced this formula to a rather simpler form:

$$d_s/d_r = 1.8\left(b/d_r\right)^{\frac{1}{4}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (12)$$

where $d_s$ is scour depth from water surface, $d_r$ is regime depth of the river, and b is the breadth of the pier projected at right angles to the direction of attacking flow.

It is interesting to note that the mean depth of flow between piers (as distinct from the local scour round them) is shown by Leopold[12] to follow the regime law of variation as the two thirds power of discharge intensity in the bridge over the Powder River at Arvada, Wyoming.

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[a] February, 1960, by Emmett M. Laursen.

[9] Prof. of Civ. Engrg. and Cons. Engr.

[10] "The Behaviour and Control of Rivers and Canals," (with the aid of models), by Sir C. C. Inglis, Govt. of India, Central Water Power Irrigation and Navigations Research Sta., Ponna, 1949.

[11] "Regime Behaviour of Canals and Rivers," by T. Blench, Butterworths Scientific Publications, London and Toronto, 1957.

[12] "The Hydraulic Geometry of Steam Channels and Some Physiographic Implications," U.S.G.S., Prof. Paper 252, 1953, Fig. 4.A.

The writer feels that Eqs. 7 and 8 should not have been derived from considerations of total load since it is bed load that is mainly effective in determining depth and that the formula is rough; furthermore, he notes that y seems to be unrelated to regime depth so is open to objection as a standard for comparison. Considering the evidence of regime theory, plus the Inglis and Leopold observations on rivers, the writer would replace Eqs. 7 and 8 by their equivalents from

$$\text{regime depth} \propto \sqrt[3]{q^2/F_b} \qquad (13)$$

and then apply geometric coefficients to various obstacles to obtain scour;[11] q is discharge intensity and $F_b$ is the appropriate bed factor.

## PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2270 is identified as 2270(ST9) which indicates that the paper is contained in the ninth issue of the Journal of the Structural Division during 1959.

### VOLUME 85 (1959)

MAY: 2014(AT2), 2015(AT2), 2016(AT2), 2017(HY5), 2018(HY5), 2019(HY5), 2020(HY5), 2021(HY5), 2022(HY5), 2023(PL2), 2024(PL2), 2025(PL2), 2026(PP1), 2027(PP1), 2028(PP1), 2029(PP1), 2030(SA3), 2031(SA3), 2032(SA3), 2033(SA3), 2034(ST5), 2035(ST5), 2036(ST5), 2037(ST5), 2038(PL2), 2039(PL2), 2040(AT2)[c], 2041(PL2)[c], 2042(PP1)[c], 2043(ST5)[c], 2044(SA3)[c], 2045(HY5)[c], 2046(PP1), 2047(PP1).

JUNE: 2048(CP1), 2049(CP1), 2050(CP1), 2051(CP1), 2052(CP1), 2053(CP1), 2054(CP1), 2055(CP1), 2056 (HY6), 2057(HY6), 2058(HY6), 2059(IR2), 2060(IR2), 2061(PO3), 2062(SM3), 2063(SM3), 2064(SM3), 2065 (ST6), 2066(WW2), 2067(WW2), 2068(WW2), 2069(WW2), 2070(WW2), 2071(WW2), 2072(CP1)[c], 2073(IR2)[c], 2074(PO3)[c], 2075(ST6)[c], 2076(HY6)[c], 2077(SM3)[c], 2078(WW2)[c].

JULY: 2079(HY7), 2080(HY7), 2081(HY7), 2082(HY7), 2083(HY7), 2084(HY7), 2085(HY7), 2086(SA4), 2087 (SA4), 2088(SA4), 2089(SA4), 2090(SA4), 2091(EM3), 2092(EM3), 2093(EM3), 2094(EM3), 2095(EM3), 2096 (EM3), 2097(HY7)[c], 2098(SA4)[c], 2099(EM3)[c], 2100(AT3), 2101(AT3), 2102(AT3), 2103(AT3), 2104(AT3), 2105(AT3), 2106(AT3), 2107(AT3), 2108(AT3), 2109(AT3), 2110(AT3), 2111(AT3), 2112(AT3), 2113(AT3), 2114(AT3), 2115(AT3), 2116(AT3), 2117(AT3), 2118(AT3), 2119(AT3), 2120(AT3), 2121(AT3), 2122(AT3), 2123(AT3), 2124(AT3), 2125(AT3).

AUGUST: 2126(HY8), 2127(HY8), 2128(HY8), 2129(HY8), 2130(PO4), 2131(PO4), 2132(PO4), 2133(PO4), 2134 (SM4), 2135(SM4), 2136(SM4), 2137(SM4), 2138(HY8)[c], 2139(PO4)[c] 2140(SM4)[c].

SEPTEMBER: 2141(CO2), 2142(CO2), 2143(CO2), 2144(HW3), 2145(HW3), 2146(HW3), 2147(HY9), 2148(HY9), 2149(HY9), 2150(HY9), 2151(IR3), 2152(ST7)[c], 2153(IR3), 2154(IR3), 2155(IR3), 2156(IR3), 2157(IR3), 2158 (IR3), 2159(IR3), 2160(IR3), 2161(SA5), 2162(SA5), 2163(ST7), 2164(ST7), 2165(SU1), 2166(SU1), 2167(WW3), 2168(WW3), 2169(WW3), 2170(WW3), 2171(WW3), 2172(WW3), 2173(WW3), 2174(WW3), 2175(WW3), 2176 (WW3), 2177(WW3), 2178(CO2)[c], 2179(IR3)[c], 2180(HW3)[c], 2181(SA5)[c], 2182(HY9)[c], 2183(SU1)[c], 2184 (WW3)[c], 2185(PP2)[c], 2186(ST7)[c], 2187(PP2), 2188(PP2).

OCTOBER: 2189(AT4), 2190(AT4), 2191(AT4), 2192(AT4), 2193(AT4), 2194(EM4), 2195(EM4), 2196(EM4), 2197(EM4), 2198(EM4), 2199(EM4), 2200(HY10), 2201(HY10), 2202(HY10), 2203(PL3), 2204(PL3), 2205 (PL3), 2206(PO5), 2207(PO5), 2208(PO5), 2209(PO5), 2210(SM5), 2211(SM5), 2212(SM5), 2213(SM5), 2214 (SM5), 2215(SM5), 2216(SM5), 2217(SM5), 2218(ST8), 2219(ST8), 2220(EM4), 2221(ST8), 2222(ST8), 2223 (ST8), 2224(HY10), 2225(HY10), 2226(PO5), 2227(PO5), 2228(PO5), 2229(ST8), 2230(EM4), 2231(EM4), 2232(AT4)[c], 2233(PL3)[c], 2234(EM4)[c], 2235(HY10)[c], 2236(SM5)[c], 2237(ST8)[c], 2238(PO5)[c], 2239(ST8), 2240 (PL3).

NOVEMBER: 2241(HY11), 2242(HY11), 2243(HY11), 2244(HY11), 2245(HY11), 2246(SA6), 2247(SA6), 2248 (SA6), 2249(SA6), 2250(SA6), 2251(SA6), 2252(SA6), 2253(SA6), 2254(SA6), 2255(SA6), 2256(ST9), 2257(ST9), 2258(ST9), 2259(ST9), 2260(HY11), 2261(ST9)[c], 2262(ST9), 2263(HY11), 2264(ST9), 2265(HY11), 2266(SA6), 2267(SA6), 2268(SA6), 2269(HY11)[c], 2270(ST9).

DECEMBER: 2271(HY12)[c], 2272(CP2), 2273(HW4), 2274(HW4), 2275(HW4), 2276(HW4), 2277(HW4), 2278 (HW4), 2279(HW4), 2280(HW4), 2281(IR4), 2282(IR4), 2283(IR4), 2284(IR4), 2285(PO6), 2286(PO6), 2287 (PO6), 2288(PO6), 2289(PO6), 2290(PO6), 2291(PO6), 2292(SM6), 2293(SM6), 2294(SM6), 2295(SM6), 2296 (SM6), 2297(WW4), 2298(WW4), 2299(WW4), 2300(WW4), 2301(WW4), 2302(WW4), 2303(WW4), 2304(HW4), 2305(ST10), 2306(CP2), 2307(CP2), 2308(ST10), 2309(CP2), 2310(HY12), 2311(HY12), 2312(PO6), 2313(PO6), 2314(ST10), 2315(HY12), 2316(HY12), 2317(HY12), 2318(WW4), 2319(SM6), 2320(SM6), 2321(ST10), 2322 (ST10), 2323(HW4)[c], 2324(CP2)[c], 2325(SM6)[c], 2326(WW4)[c], 2327(IR4)[c], 2328(PO6)[c], 2329(ST10)[c], 2330 (CP2).

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JANUARY: 2331(EM1), 2332(EM1), 2333(EM1), 2334(EM1), 2335(HY1), 2336(HY1), 2337(EM1), 2338(EM1), 2339(HY1), 2340(HY1), 2341(SA1), 2342(EM1), 2343(SA1), 2344(ST1), 2345(ST1), 2346(ST1), 2347(ST1), 2348(EM1)[c], 2349(HY1)[c], 2350(ST1), 2351(ST1), 2352(SA1)[c], 2353(ST1)[c], 2354(ST1).

FEBRUARY: 2355(CO1), 2356(CO1), 2357(CO1), 2358(CO1), 2359(CO1), 2360(CO1), 2361(PO1), 2362(HY2), 2363(ST2), 2364(HY2), 2365(SU1), 2366(HY2), 2367(SU1), 2368(SM1), 2369(HY2), 2370(SU1), 2371(HY2), 2372(PO1), 2373(SM1), 2374(HY2), 2375(PO1), 2376(HY2), 2377(CO1)[c], 2378(SU1), 2379(SU1), 2380(SU1), 2381(HY2)[c], 2382(ST2), 2383(SU1), 2384(ST2), 2385(SU1)[c], 2386(SU1), 2387(SU1), 2388(SU1), 2389(SM1), 2390(ST2)[c], 2391(SM1)[c], 2392(PO1)[c].

MARCH: 2393(IR1), 2394(IR1), 2395(IR1), 2396(IR1), 2397(IR1), 2398(IR1), 2399(IR1), 2400(IR1), 2401(IR1), 2402(IR1), 2403(IR1), 2404(IR1), 2405(IR1), 2406(IR1), 2407(SA2), 2408(SA2), 2409(HY3), 2410(ST3), 2411 (SA2), 2412(HW1), 2413(WW1), 2414(WW1), 2415(HY3), 2416(HW1), 2417(HW3), 2418(HW1)[c], 2419(WW1)[c], 2420(WW1), 2421(WW1), 2422(WW1), 2423(WW1), 2424(SA2), 2425(SA2)[c], 2426(HY3)[c], 2427(ST3)[c].

APRIL: 2428(ST4), 2429(HY4), 2430(PO2), 2431(SM2), 2432(PO2), 2433(ST4), 2434(EM2), 2435(PO2), 2436 (ST4), 2437(ST4), 2438(HY4), 2439(EM2), 2440(EM2), 2441(ST4), 2442(SM2), 2443(HY4), 2444(ST4), 2445 (EM2), 2446(ST4), 2447(EM2), 2448(SM2), 2449(HY4), 2450(ST4), 2451(HY4), 2452(HY4), 2453(EM2), 2454 (EM2), 2455(EM2)[c], 2456(HY4)[c], 2457(PO2)[c], 2458(ST4)[c], 2459(SM2)[c].

MAY: 2460(AT1), 2461(ST5), 2462(AT1), 2463(AT1), 2464(CP1), 2465(CP1), 2466(AT1), 2467(AT1), 2468(SA3), 2469(HY5), 2470(ST5), 2471(SA3), 2472(SA3), 2473(ST5), 2474(SA3), 2475(ST5), 2476(SA3), 2477(ST5), 2478 (HY5), 2479(SA3), 2480(ST5), 2481(SA3), 2482(CO2), 2483(CO2), 2484(HY5), 2485(HY5), 2486(AT1)[c], 2487 (CP1)[c], 2488(CO2)[c], 2489(HY5)[c], 2490(SA3)[c], 2491(ST5)[c], 2492(CP1), 2493(CO2).

[c] Discussion of several papers, grouped by divisions.